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EXPERIMENTAL PAVED SHOULDERS ON FROST SUSCEPTIBLE SOILS

IHR-404
FINAL REPORT

by

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by

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ABSTRACT

Four-paved-shoulder types were included experimentally in a construction project that also included continuously reinforced concrete pavement. The shoulder types were a bituminous-aggregate mixture (BAM) that served as both base and wearing surface; a cement-aggregate mixture (CAM) and a pozzolan-aggregate mixture (PAM) surfaced with bituminous concrete; and paving grade portland cement concrete (PCC). The BAM, CAM and PAM mixtures also were used as pavement subbase.

The construction took place in an area of frost-susceptible fine-grained soils that had been associated previously in Illinois with severe winter heave of the CAM and PAM shoulder types and concurrent freeze-thaw disintegration and structural failure. Durability improvements were made in the new CAM and PAM mixtures.

Three years of service experience showed the improved CAM and PAM mixtures still to be susceptible to freeze-thaw damage in shoulder base construction, but to serve adequately as pavement subbase. The service afforded by the BAM shoulder type was disappointing and at variance with previous experience in Illinois. Construction aberrations are believed to be a principal cause of the poor service. The BAM material was found to serve adequately as pavement subbase. The PCC shoulders were still giving excellent service at the conclusion of observations. The presence and absence of shoulder subbase and of sealant in the pavement-shoulder joint were other variables studied. Detailed information useful in selecting shoulder paving materials and structural designs is presented.

Contrary to experience elsewhere in northern Illinois, significant differential vertical pavement-shoulder movements did not occur during the period of the study. A change from unstabilized to stabilized aggregate pavement subbase is believed responsible for the difference.

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EXPERIMENTAL PAVED SHOULDERS ON FROST SUSCEPTIBLE SOILS

INTRODUCTION

Shoulders that can receive vehicles from the traffic lane at normal operating speeds and permit them to return safely to the pavement are essential features of modern highways. For high-speed, heavily-traveled highways, such as those of the Interstate system, this requires that the shoulders be durable and capable of supporting fairly frequent incursions under all conditions of weather.

Initially, a shoulder consisting of a granular base covered with a bituminous surface treatment was believed adequate for high-speed and high-volume travel conditions in Illinois. Experience soon made it apparent, however, that these shoulders lacked all-weather stability, and that the maintenance task was a serious problem. The substitution of a stabilized mixture of granular material cemented with asphalt, portland cement, or lime-flyash for base construction, and the substitution of a bituminous mat for the seal coat, were found to correct the situation in most instances, but not in all. The major problem in these latter instances was a differential heave of the shoulder with respect to the pavement and subsequent deterioration of the shoulder structure. Lateral dislocation of the shoulder from the pavement also commonly occurred in these instances.

Typical of the paved shoulders giving poor service were those of an expressway entering Chicago from the southwest (1).[1/] Soon after the opening of this expressway in the fall of 1964, an extreme upward displacement of the paved shoulder with respect to the adjacent conventionally reinforced and continuously reinforced portland cement concrete pavements was noted. The vertical displacement was accompanied by some lateral displacement, by the frequent occurrence of longitudinal

1/ Numerals in parentheses and underlined refer to references listed at conclusion of report.

cracks about one foot from the pavement edge, and by a considerable amount of random cracking between the longitudinal cracks and the interface of the pavement and shoulder.

Most of the pavement was placed on a six-inch subbase of well-graded crushed stone which was extended through the shoulder areas to the sod cover on the side slopes. Lesser use was made of a trenched subbase of the same material, and minor use was made of trenched subbase in combination with pipe underdrain.

Three kinds of material were used in the shoulder base courses on this portion of the expressway: (1) cement-aggregate mixture (CAM); (2) pozzolan-aggregate mixture (PAM); and (3) bituminous-aggregate mixture (BAM). The CAM and PAM bases were surfaced with bituminous concrete; the BAM base was not. Except for some localized small vertical and horizontal displacements that occurred in the BAM base, the deficiencies were confined to locations where the CAM and PAM shoulder bases had been used. A study of the conditions surrounding the paved shoulder failures indicated that the displacement and attendant distress originated through the exposure of frost-susceptible and expansive materials to excessive amounts of surface water. Several factors seemed to have acted either in combination or separately to aggravate the condition, among them being: (1) an embankment soil especially susceptible to frost expansion when exposed to large quantities of water and also capable of expansion when exposed to moisture; (2) a pavement subbase material somewhat capable of frost expansion when exposed to water but also capable of serving as a source of free water to be drawn upon by contiguous frost-susceptible materials; and (3) base materials lacking adequate durability when exposed to freeze-thaw cycles in the presence of water or brine.

The value of the findings from this investigation of shoulder displacement and deterioration after failure had taken place was limited by the lack of adequate knowledge of many details regarding conditions prevailing prior to the occurrence of the failures. The study that is the subject of the present report was undertaken to provide this sort of information as well as information on the behavior of paved shoulders in service.

In the present study, experimentally designed paved shoulders were included in a construction contract that also included continuously reinforced concrete pavement on Interstate 80 immediately east of Joliet, Illinois. The construction is identified as Section 99-4-1, Project I-80-4(139)135, Will County, District 1, and located as shown in Figure 1. Construction work was completed during the 1967 season, with the exception of a few minor items including a portion of the joint sealing, final grading of shoulders and roadside slopes, the application of topsoil, and seeding. Sealing of the pavement-shoulder joint was completed on the eastbound lanes in the fall of 1967, and in the westbound lanes in the spring of 1968. The roadway was opened to traffic in January 1968, and all remaining construction work completed in that year.

Numerous items of instrumentation were placed during construction for detailed observations of shoulder behavior. Research observations were made during and following construction.

The study has yielded some important information to date; however, it must be recognized that behavioral projections over what might be a 20-year life span, based on less than four years of service, may be subject to modification as further experience accumulates.

OBJECTIVES

The major objective of this research was to develop definitive information that would permit the selection from among alternative shoulder pavement designs

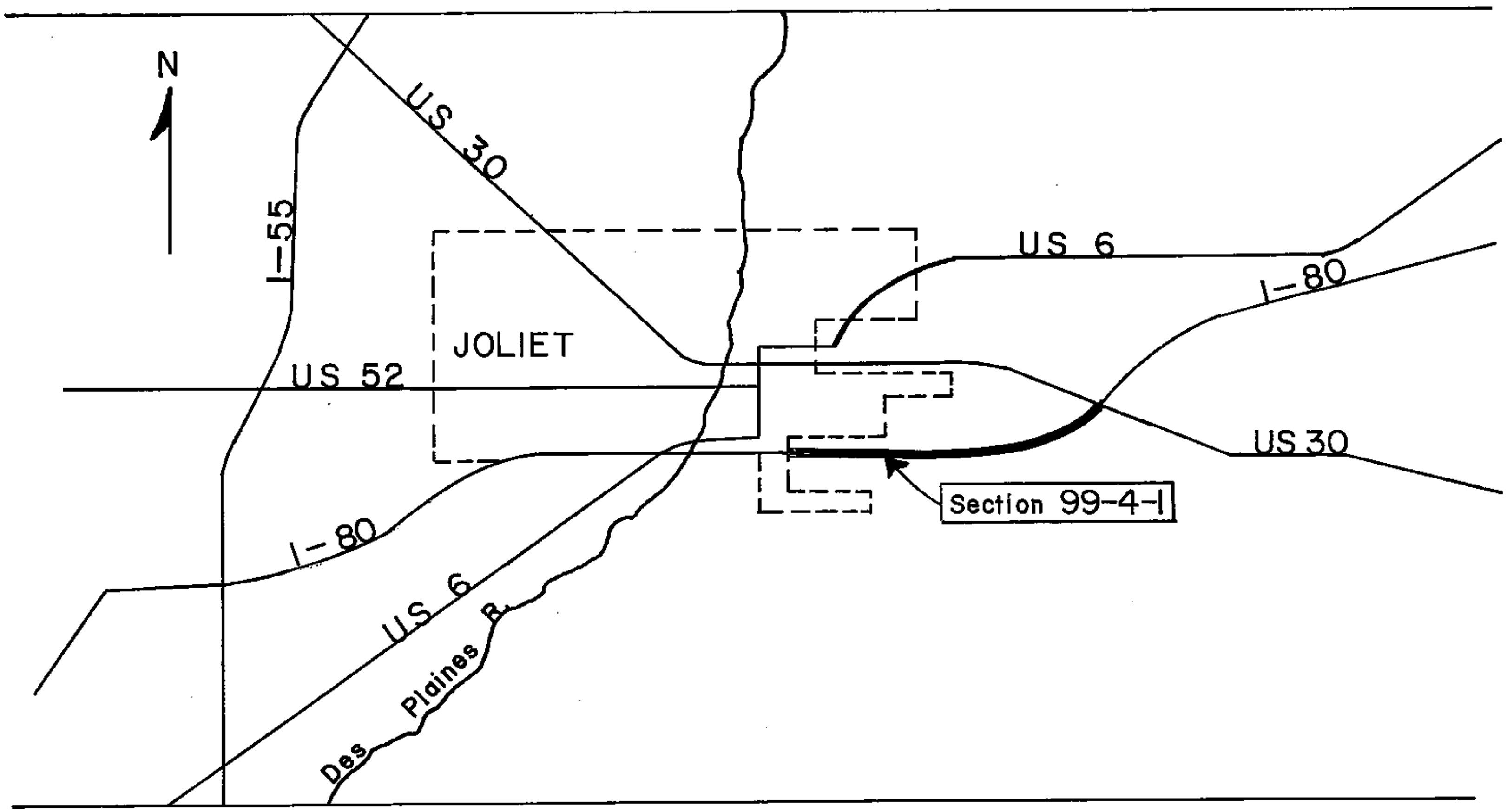


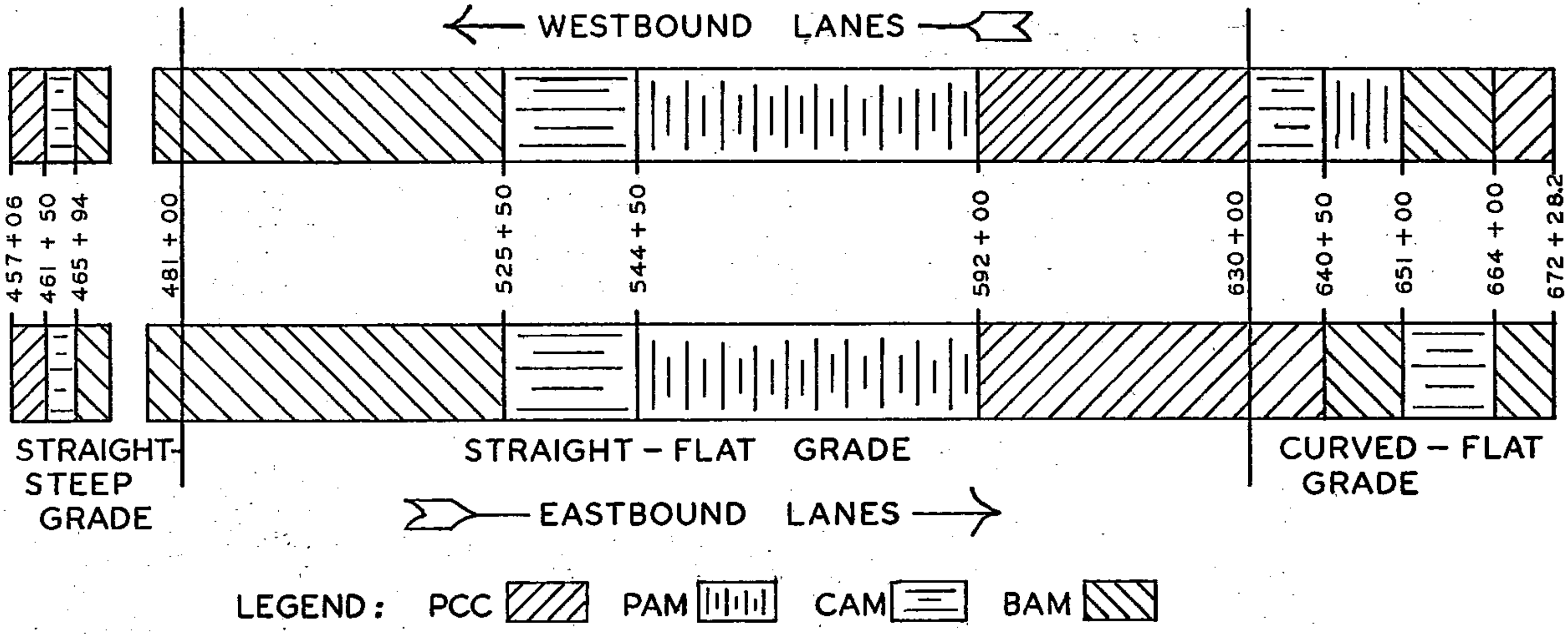
FIGURE I. Location of Experimental Project.

and materials, those that will afford the best service and overall economy of construction and maintenance. A secondary objective was the development of additional information on the interactions of embankment soils, frost, moisture and deicing operations with shoulder materials and shoulder designs, to help understand the underlying causes of the dislocations and disintegrations that have occurred in the past.

EXPERIMENTAL LAYOUT

Almost all of the 3.9-mile length of construction Section 99-4-1 was included in the experiment. Because of some differences in horizontal and vertical alignments, the individual test sections were placed in three alignment groupings. Group I, at the west end, is within an area of straight horizontal alignment with a fairly steep vertical gradient (up to 3 percent upgrade). Group II, in the central part, is within an area of straight alignment and flat grade. Group III, at the east end, is in an area of horizontal curvature and flat grade where the pavement is superelevated and the crown removed, but with no change in the shoulder slopes. Layout details are shown in Figure 2.

The principal variables in the experimentation are the shoulder base materials, shoulder subbase materials and the use and nonuse of shoulder subbase, and the presence and absence of sealant at the pavement-shoulder interface. Four material types were used in the shoulder bases: (1) bituminous-aggregate mixture (BAM); (2) cement-aggregate mixture (CAM); (3) pozzolan-aggregate mixture (PAM); and (4) portland cement concrete (PCC). A bituminous-concrete surfacing course was placed on the CAM and PAM bases; the BAM and PCC bases were constructed to serve traffic without additional surfacing. Three types of open-graded aggregates of differing gradation were used as shoulder subbases, and some sections were



(SCALE: 1 INCH = 2200 FEET)

I-80, SECTION 99-4-1, WILL COUNTY

FIGURE 2. Location of Shoulder Types in Experimentation

constructed without subbase. Material details are presented in a following section of the report.

The subbase underlying the continuously reinforced concrete pavement was constructed of the same materials as used in the adjacent shoulder bases, except that CAM was used in the pavement subbase adjacent to the PCC shoulder structures.

The portland cement concrete shoulders of this project were the third phase of a more comprehensive study of the potential of portland cement concrete as a shoulder paving material. Details of this study are reported elsewhere (2).

All of the test combinations are included at least once in alignment Group II, and as many of the combinations as possible are repeated in Groups I and III. The test sections vary in length from about 450 feet to over 1600 feet. The layout was planned to facilitate construction as much as possible, particularly with reference to placement of materials. The locations of the shoulder types are shown in Figure 2. The makeup of each test section, together with the location of the instrumentation, is shown in Table 1.

Instrumentation was installed during construction to provide data on vertical pavement and shoulder movements and on frost penetration. Embankment soil moisture contents and densities were measured by the nuclear method. Additional supplemental information was obtained through condition surveys and coring studies to help evaluate the performance of the shoulders.

SOILS

The embankment materials at the experiment site are mostly fine-textured A-4, A-6, and A-7-6 soils. The physical characteristics shown in Table 2 for samples taken at settlement plate locations are representative of those of the soils throughout the project. The material is principally glacial drift of

TABLE 1

TEST SECTIONS AND INSTRUMENT LOCATIONS

Test Sections From	Test Sections To	Joint Sealed	Shoulder Type 1/ Eastbound	Shoulder Type 1/ Westbound	Instrumentation Station	Instrumentation Type 2/
			Alignment Group I			
457+06	459+28	Yes	PCC/B	PCC/E		
459+28	461+50	No	"	"		
461+50	463+72	Yes	CAM/B	CAM/E	462+50	S.P.
463+72	465+94	No	"	"	464+50	S.P.
465+94	468+16	Yes	BAM/B	BAM/E		
468+16	470+38	No	"	"		
470+38	476+34		Rowell Avenue Bridge			
476+34	479+67	Yes	BAM/B	BAM/E		
478+67	481+00	No	"	"		
			Alignment Group II			
481+00	493+50	Yes	BAM/B	BAM/E	487+00	S.P.
493+50	506+00	No	"	"	495+00	S.P., F.G.
506+00	515+75	Yes	BAM/C	"		
515+75	525+50	No	"	BAM/A		
525+50	530+25	Yes	CAM/C	CAM/A		
530+25	535+00	No	"	"		
535+00	539+75	Yes	CAM/B	CAM/E	537+50	S.P.
539+75	544+50	No	"	"	542+00	S.P.
544+50	551+75	Yes	PAM/B	PAM/E	548+00	S.P., F.G.
551+75	559+00	No	"	"		
559+00	575+50	Yes	PAM/C	PAM/A	553+50	S.P.
575+50	592+00	No	"	"		
592+00	603+50	Yes	PCC/C	PCC/A	619+00	S.P.
603+50	615+00	No	"	"		
615+00	622+50	Yes	PCC/B	PCC/E		
622+50	630+00	No	"	"	626+00	S.P., F.G.
			Alignment Group III			
630+00	632+62	Yes	PCC/B	CAM/E		
632+62	635+20	No	"	"		
635+20	637+87	Yes	PCC/E	CAM/B		
637+87	640+50	No	"	"		
640+50	643+12	Yes	BAM/E	PAM/B		
643+12	645+75	No	"	"		
645+75	648+37	Yes	BAM/B	PAM/E	647+00	S.P.
648+37	651+00	No	"	"	649+50	S.P.
651+00	653+62	Yes	CAM/R	BAM/R		
653+62	656+25	No	"	"		
656+25	660+12	Yes	CAM/B	BAM/B		
660+12	664+00	No	"	"		
664+00	666+00	Yes	BAM/E	PCC/B		
666+00	668+00	No	"	"		
668+00	670+14	Yes	BAM/B	PCC/E		
670+14	672+28	No	"	BAM/E		

1/ Single letter indicates the type of subbase (A, B, or C), earth subgrade (E), or crushed stone over bedrock (R).

2/ S.P.= settlement plate installation, eastbound and westbound;
F.G.= frost gage installation, eastbound and westbound.

TABLE 2

PHYSICAL PROPERTIES OF SOILS AT SETTLEMENT PLATE LOCATIONS

Station	Eastbound					Westbound				
	Clay (less than .005 mm) (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index	AASHO Soil Class	Clay (less than .005 mm) (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index	AASHO Soil Class
462+50	22	29	14	15	A-6(3)	27/51	28/35	16/20	12/15	A-6(4)/A-6(10)
464+50	38	31	21	10	A-4(8)	51	35	20	15	A-6(10)
487+00	48	43	13	30	A-7-6(16)	50	41	29	12	A-7-6(10)
495+00	57	41	21	20	A-7-6(12)	50/53	45/36	26/21	19/15	A-7-6(12)/A-6(10)
537+50	42	33	18	15	A-6(10)	41	31	20	11	A-6(9)
542+00	40	31	15	16	A-6(10)	47	38	17	21	A-6(13)
548+00	46	32	19	13	A-6(8)	43	37	17	20	A-6(12)
553+50	36	31	17	14	A-6(9)	47	33	18	15	A-6(10)
619+00	56/50	49/37	22/26	27/11	A-7-6(16)/A-6(11)	54	35	17	18	A-6(11)
626+00	50/42	38/40	21/15	17/25	A-6(12)/A-7-6(14)	49	39	14	25	A-6(14)
647+00	58	29	14	15	A-6(10)	50	31	17	14	A-6(10)
649+50	14	15	11	4	A-4(3)	29	26	14	12	A-6(7)

NOTE: 1. Double number means two soil layers.
2. Average of the plastic limits is 18.3 percent.

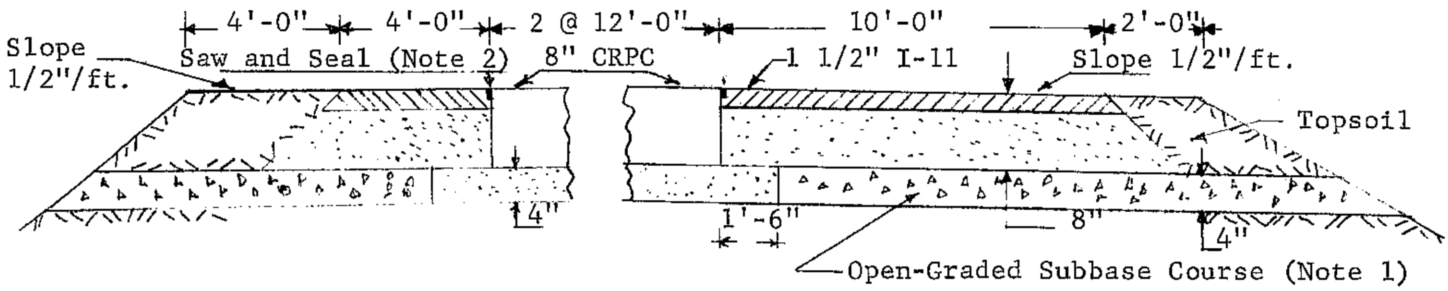
Wisconsinan Age (3) overlying dolomitic limestone bedrock. Although the glacial drift is relatively thin in the area, the bedrock appeared in the zone of construction operations in only one instance. This is between Stations 654 and 664 where the pavement structure inclusive of subbase was constructed on a foot or more of crushed stone placed directly on bedrock.

These soils are representative of those that have been in the past adversely affected by freeze-thaw action in Illinois.

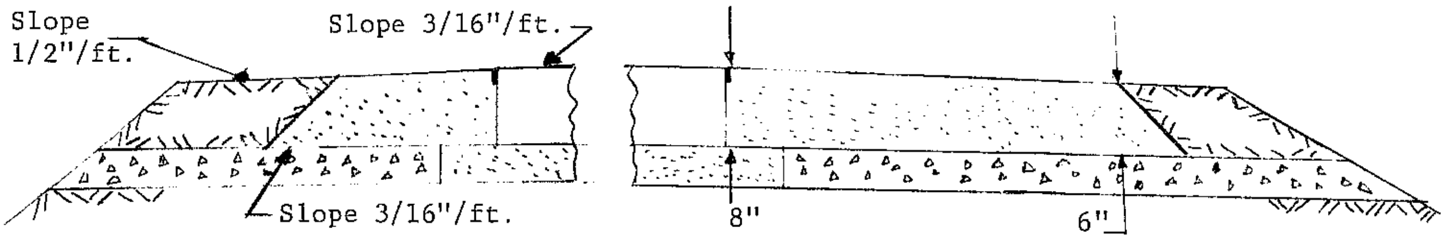
STRUCTURAL DESIGN OF SHOULDERS

The structural components of the experimental shoulders and their dimensions are shown in Figure 3. The outside paved shoulders are 10 feet wide and the median shoulders are 4 feet wide. The CAM and PAM base courses are of uniform thickness (6 1/2 inches) surfaced with 1 1/2 inches of bituminous concrete. The BAM and PCC shoulders taper from 8 inches thick at the pavement edge to a minimum of 6 inches at the outside edge. The 4-inch thick open-graded shoulder subbase, where used, extends from the edge of the pavement subbase to the shoulder slope. It is left open on the shoulder slope to facilitate free drainage. The shoulders are sloped at the rate of 1/2 inch per foot. The PCC shoulders are tied to the pavement, have 6-foot-long rumble strips at 60-foot intervals, and dummy-groove transverse joints at 20-foot intervals. These and other features are shown in Figure 3.

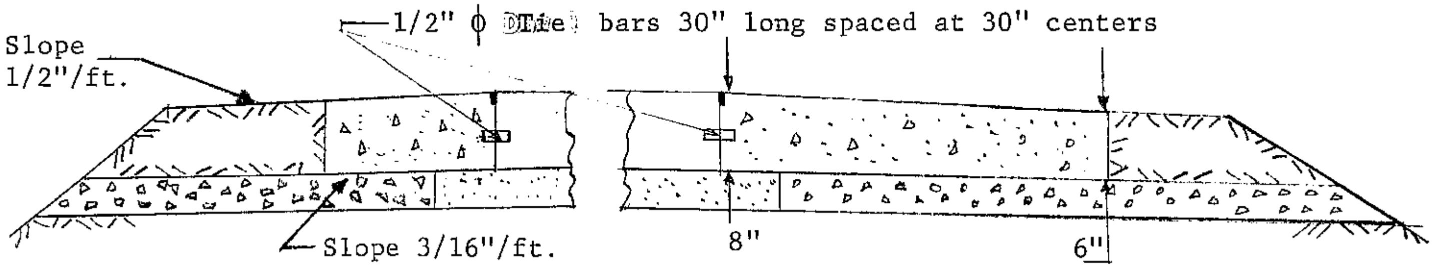
The mainline pavement in the test area is continuously reinforced portland cement concrete, 8 inches thick, with a 4-inch thickness of subbase of the same kind of material as that used in the adjoining shoulder, except that CAM was used as the pavement subbase in locations adjacent to the portland cement concrete shoulders.



CAM or PAM



BAM

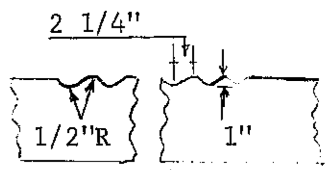


PCC

Note 1. Open-Graded Subbase Course used on approximately one half of test sections; shoulder structure placed on regular embankment soil elsewhere.

Note 2. Shoulder-pavement joint sawed to depth of 3/4" and width of 1/2" and sealed with hot-poured rubber asphalt joint-sealing compound on one half of each test section.

Note 3. Rumble strips on PCC shoulders 6'-0" long, full width, at 60' spacing. Dummy-groove transverse joint every 20'.



Rumble Strip Design

Figure 3. Design of experimental shoulders

SHOULDER MATERIALS

Bituminous-aggregate mixture (BAM).--The bituminous-aggregate mixture that was used in the shoulder and pavement subbase construction is a dense-graded material composed of crushed stone and paving asphalt (Grade 200-300). The mix was designed to contain 4.5 percent asphalt by weight of the total mixture, except that 5.0 percent asphalt was to be used in the shoulder surface lift.

Gradation specification limits under which the aggregate was furnished, and a typical aggregate gradation, are as follows:

Sieve Size	Specification Limits	Typical Gradation
	(percent passing)	
1 in.	100	100
1/2 in.	60-100	82
No. 4	40-65	48
No. 8	25-55	35
No. 200	5-15	8

The minus No. 40 sieve fraction had little or no plasticity.

The mixture was prepared in a hot-mix plant and delivered to the jobsite at temperatures specified to range between 225^{o}F and 325^{o}F.

Marshall tests of a sample of the bituminous-aggregate mixture containing 4.5 percent asphalt taken at the jobsite produced the following results:

Stability (lb.)	2485
Flow (1/100 in.)	11

Cement-aggregate mixture (CAM).--The cement-aggregate mixture used both as pavement subbase and as shoulder base was composed of Type 1 cement and a blend of crushed stone and stone screenings. Specification limits for the gradation of the aggregate and a typical gradation of the material furnished to meet these limits are as follows:

Sieve Size	Specification Limits	Typical Gradation
	(percent passing)	
1 in.	100	100
1/2 in.	60-100	87
No. 4	55-75	57
No. 8	40-65	42
No. 200	5-12	10

Blending stone screenings with the available crushed stone material was necessary to produce the desired fineness at the No. 4 and No. 8 sieve levels. Previous experience had shown that mixtures of coarser gradation lacked durability at normal cement contents.

A cement content of 5.2 percent of the dry weight of the aggregate was established for the mixture based on standard short-cut soil cement mixture design procedures. At 4, 6, and 8 percent cement by weight, laboratory 7-day compressive strengths of 648, 1460, and 2727 psi respectively were recorded. The maximum density of the mixture was determined to be 142 pcf at an optimum moisture content of 7.5 percent under AASHO T99 (Method C).

The mixture was prepared in a batch-type mixer at a central mixing plant and trucked to the jobsite.

Pozzolan-aggregate mixture (PAM).--Wet-bottom boiler slag was combined with lime and flyash (and water) to produce the pozzolan-aggregate mixture.

Specification limits for the boiler slag, and a typical gradation, are as follows:

Sieve Size	Specification Limits	Typical Gradation
	(percent passing)	
No. 4	80-100	97
No. 10	55-90	77
No. 40	0-25	13
No. 200	0-10	3

The flyash was specified to meet the requirements of ASTM Designations C 379 or C 342 with an allowable 10 percent maximum loss on ignition. At the time of mixing, the flyash was required to meet the following gradation requirements in dry sieving:

Sieve Size	Minimum Passing (percent)
1/2 in.	100
3/8 in.	95
No. 10	75

The moisture content of the dampened flyash at mixing was required not to exceed 35 percent.

Both the slag and flyash were furnished from a nearby power plant.

The lime was furnished to comply with the requirements of ASTM Designation: C 207, "Hydrated Lime for Masonry Purposes, Type N," except that there was to be a minimum of 90 percent total calcium and magnesium oxides (non-volatile basis), and the calcium oxide and magnesium contents on an as-received basis were not to exceed 5 percent.

The pozzolan-aggregate mixture composition specification, and the job mix formula were as follows:

Component	Proportion	
	Specification (percent)	Job-Mix Formula (percent)
Slag	56-78	73.0 ± 2.0
Flyash	15-30	24.0 ± 1.5
Lime	2-4	3.0 ± 0.3

The mixture, meeting the job-mix formula, had an optimum moisture content of 9 percent and a maximum dry density of 135 pcf. It was tested in accordance with AASHO Designation T 180 (Method C), except that three lifts were used instead of five. The moisture content of the mixture at compaction was required to be within 85 to 110 percent of this moisture content.

In designing the mixture, test cylinders were prepared at optimum moisture content and compacted in accordance with AASHO Designation T 180 (Method C) with the previously described modification. Mixtures with lime contents of 2, 3, and 4 percent were used in the preparation of the cylinders. The specimens at the various lime percentages were placed in watertight containers immediately after molding and heated to $100^{o}F \pm 3^{o}$ for 7 days. At the end of 7 days, the test cylinders were removed from the container, allowed to cool to room temperature, and soaked in water for 4 hours. After soaking, they were allowed to drain, were capped, and then broken to determine compressive strengths. The design lime content was established to meet a requirement that the minimum average compressive strength be no lower than 400 psi and that no individual test be lower than 300 psi.

The materials were mixed at a central mixing plant (pugmill type) and trucked to the jobsite for spreading and compacting.

Portland cement concrete (PCC).--The portland cement concrete mixture for the PCC shoulder was specified to meet the requirements of the Illinois standard specifications for portland cement concrete pavement construction. Type 1A cement was used. The materials in proportion were combined with water in a central mixing plant and wet-batched to the jobsite in trucks. Additional details are reported in reference (2).

Bituminous concrete surfacing (Class I).--A 1 1/2-inch surface course of bituminous concrete mixture, fine dense-graded aggregate type, was placed over the CAM and PAM shoulder bases. The mixture, consisting of crushed limestone coarse aggregate, sand, limestone dust mineral filler, and a paving grade asphalt, was required to meet the Illinois standard specifications for Class I bituminous concrete.

Shoulder subbase materials.--The experiment provided for the shoulder bases to be placed on subbases composed of aggregates furnished under three differing gradation specifications, as well as on the natural subgrade. The three gradation specifications, all commonly used for other purposes in Illinois, and typical analyses of materials furnished under them, are shown in Table 3.

The intent of the subbase material study was to determine which of the three materials furnished under the three differing gradation specifications would provide the best combination of manageability during construction and drainability. A washed sand very similar to that used in concrete mixtures was furnished to meet the Type A specification as shown in Table 3. Crushed stones were furnished to meet the Type B and Type C specifications. The Type B specification was the Illinois specification for coarse aggregate for structural concrete. The Type C specification was the Illinois specification for the smaller of two sizes of aggregate (Size B) for paving concrete.

Shoulder joint sealant.--The sealant used at locations where the experimentation required cutting and sealing the joint at the pavement-shoulder interface was a hot-poured rubber-asphalt compound typical of those meeting high-standard specifications with respect to bond, resilience, and impact. This material was used also in the transverse joints of the PCC shoulders.

EMBANKMENT CONSTRUCTION

Embankment construction on the experimental project began in September 1966 and was mostly completed prior to the close of the 1966 construction season. The remaining embankment work, consisting of the completion of a high fill west of the Rowell Avenue bridge at the west end of the project and a portion of the

TABLE 3

SPECIFIED GRADATION LIMITS AND TYPICAL
SIEVE ANALYSES FOR SHOULDER SUBBASE AGGREGATES

	Subbase Material					
	Type A		Type B		Type C	
Sieve Size	Spec. Limits (percent passing)	Typical Analysis (percent passing)	Spec. Limits (percent passing)	Typical Analysis (percent passing)	Spec. Limits (percent passing)	Typical Analysis (percent passing)
1 1/2 in.	-	-	97-100	100	100	100
1 in.	100	-	60-95	95	90-100	99
1/2 in.	90-100	100	10-30	23	25-60	40
No. 4	50-100	99	0-5	4	0-10	8
No. 16	30-80	61	-	-	-	-
No. 50	0-20	15	-	-	-	-
No. 200	0-3	3	-	-	-	-

westbound embankment between Stations 476+50 and 520+00, was completed early in 1967. The rough grade was left slightly higher than final subgrade elevation. Standard construction controls were exercised during embankment construction.

In addition to the regular moisture-density control tests made during embankment construction, moisture and density measurements also were made in the upper one-half foot of the embankment just prior to undertaking construction of the shoulder structures. The results of these tests are tabulated in Table 4. It will be noted in the table that in many instances the fine-grained soils on which the shoulder structures were placed had moisture contents well below optimum, but relatively high densities, at the time these structures were placed.

PAVEMENT SUBBASE CONSTRUCTION

Construction of the stabilized subbases for the pavement began in June 1967. This work was carried on concurrently with fine grading of the earth subgrade. Both fine grading of the subgrade and placement of the stabilized subbase material were done with a CMI Autograde which incorporates automatic control of both its vertical and horizontal movements. A hopper is attached to the front of the machine to receive material for spreading.

The prepared subbase mixtures (BAM, CAM and PAM) were unloaded directly into the CMI machine hopper from the trucks delivering them from the central plants where they were produced. They were then spread by the CMI machine to the required width and to slightly greater than the required compacted thickness. Compaction equipment followed directly behind the spreader.

After a convenient length of subbase material was placed and compacted, the CMI Autograde was backed up and then moved forward to trim the surface at the required grade ($\pm$ 1/8 in.). The length of subbase that could be constructed

TABLE 4

SHOULDER MOISTURE-DENSITY

Station	Shoulder Design 1/	Optimum Moisture	Maximum Density	In-Place Moisture	In-Place Density
		(%)	(pcf)	(%)	(pcf)
		Eastbound			
461+50	PCC/B	13.6	119.9	12.9	120.3
465+94	CAM/B	-	-	8.7	121.3
470+38	BAM/B	-	-	11.7	124.0
506+00	BAM/B	-	-	9.5	124.5
525+50	BAM/C	-	-	8.3	120.7
535+00	CAM/C	-	-	6.5	127.9
544+50	CAM/B	12.7	121.1	7.8	126.5
559+00	PAM/B	-	-	11.6	117.8
592+00	PAM/C	13.2	120.0	10.4	119.9
615+00	PCC/C	12.7	122.1	9.4	117.5
635+25	PCC/B	-	-	9.7	116.7
640+50	PCC/E	26.2	90.2	10.5	114.5
645+75	BAM/E	-	-	7.6	118.5
651+00	BAM/B	-	-	7.8	118.5
656+25	CAM/B	-	-	8.7	117.8
664+00	CAM/E	-	-	7.2	123.3
668+00	BAM/E	-	-	11.2	113.8
		Westbound			
461+50	PCC/E	-	-	12.2	122.8
465+94	CAM/E	11.6	124.8	8.7	123.3
470+48	BAM/E	-	-	10.7	120.0
515+00	BAM/E	12.2	122.9	6.8	117.0
525+50	BAM/A	-	-	6.6	126.6
535+00	CAM/A	-	-	6.6	127.6
544+50	CAM/E	-	-	7.9	119.4
559+00	PAM/E	-	-	6.5	120.6
592+00	PAM/A	13.8	117.8	10.4	120.6
615+00	PCC/A	-	-	9.5	115.0
630+00	PCC/E	15.8	112.7	9.1	120.4
641+50	CAM/B	-	-	8.8	120.5
645+75	PAM/B	-	-	10.0	115.0
651+00	PAM/E	-	-	6.6	118.3
656+25	BAM/E	-	-	5.3	123.8
664+00	BAM/B	-	-	5.7	124.6
668+00	PCC/B	12.0	125.1	7.8	125.8
672+28	PCC/E	-	-	6.4	128.5

1/ Letters A, B, and C are subbase types; letter E designates earth subgrade.

between trimming operations was limited by the requirements of the specifications and by the characteristics of the subbase materials. Specifications for the CAM required that it be placed and compaction started within one hour following the addition of water to the mix, and that compaction be completed within two hours after the addition of water, with shaping of the surface to take place near the completion of initial compaction. It was also required that uncompacted and unfinished mixtures not remain undisturbed for more than 30 minutes. To fulfill these requirements, the mixing plant and truck operations were halted several times during the day while the CMI machine was backed and moved forward in the shaping operation. The BAM and PAM materials were shaped once each day for the entire length of placement toward the close of work for the day.

Fine grading and subbase placement operations were interrupted by rainy weather a number of times. Softening of the completed earth subgrade at the outer margins of the work area on which the CMI machine was dependent for support contributed significantly to the down time.

Most of the fine grading and stabilized subbase construction was completed by the latter part of July 1967. The final section of subbase was placed west of the Rowell Avenue bridge in September 1967. The September 15 cutoff date for the use of PAM was waived and extended to October 1 to permit placing PAM in this area.

SHOULDER CONSTRUCTION

Shoulder construction took place during the period of August-November 1967, overlapping slightly the construction of the continuously reinforced concrete pavement during July and August 1967.

The earth subgrade was prepared to receive the shoulder materials by shaping with a motor grader, compacting with rollers, and reshaping with the grader as necessary to meet final grade (Figure 4).

Shoulder subbase construction.--The subbase materials where used were placed in windrows on the subgrade by trucks equipped with bottom dumps. The aggregates were spread from the windrows with a motor grader and compacted with a rubber-tired roller. The Type A (washed sand) material required the addition of moisture for adequate compaction. Final trimming of this material was accomplished with a blade operated from the pavement (Figure 5). The Type A material washed badly during rains and needed much reworking before the base material was placed on it (Figure 6). The Type B and Type C crushed stone aggregates consolidated little during compaction, and required no reworking prior to base construction.

Shoulder base construction.--The BAM, CAM, and PAM for shoulder base were placed by machines operating on the pavement (Figures 7 and 8). They were placed in two lifts of approximately equal thickness.

Each BAM lift was compacted with a rubber-tired roller followed by a steel-wheeled roller. The lower lift was compacted to meet a specified minimum requirement of 85 percent of theoretical density; the upper lift to meet a specified minimum of 90 percent of theoretical density.

Each of the two layers of CAM was compacted with a steel-wheeled roller followed by a vibratory compactor (Figure 9). The surface of the lower lift was scarified prior to placing of the upper lift, and maintained in a moist condition until the upper lift could be placed. Compaction to at least 94 percent of standard dry density within two hours of the addition of water was specified for the CAM. All work within a section of CAM placed was required to be completed within a day's

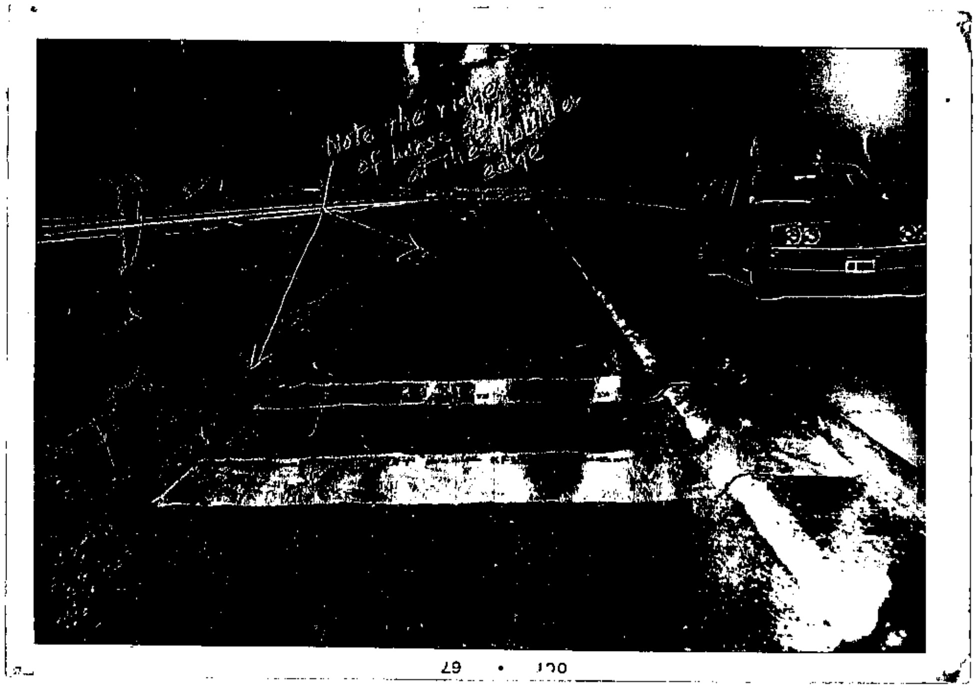


Figure 4. Finished subgrade ready to receive subbase (canvas strips in foreground are for sampling purposes).

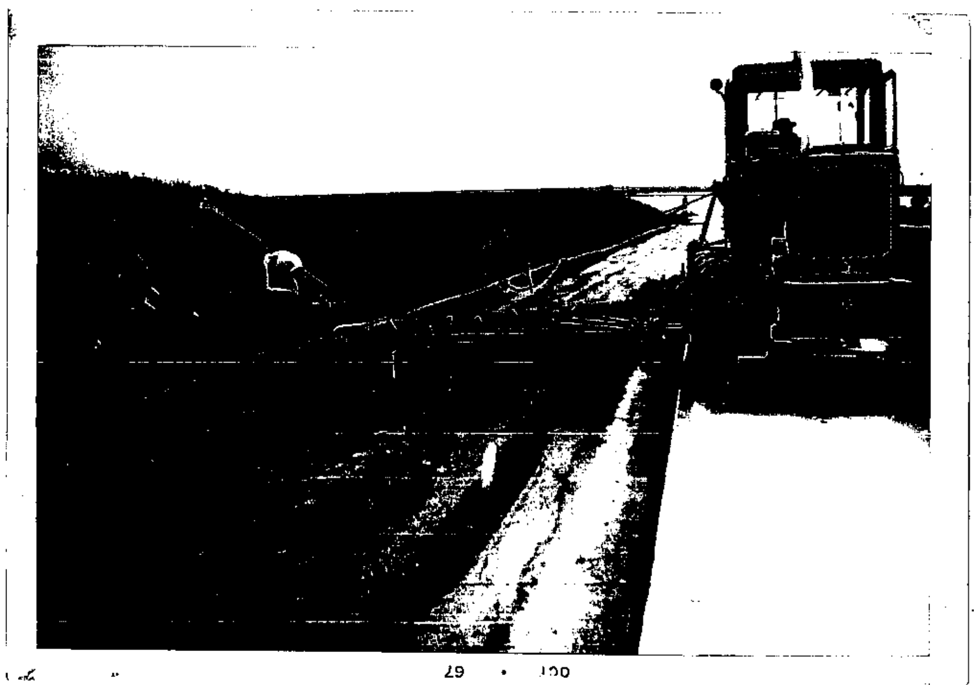


Figure 5. Grading Type A aggregate in preparation for receiving the base course.

Figure 6. Type A aggregate tended to saturate with rainfall and wash during rains.

Figure 7. Shoulder paving machines operated on the pavement.

Figure 8. Placing the top lift of the CAM base course.

Figure 9. Compacting the final CAM base lift.

time, including final shaping and compaction. It also was required that uncompacted and unfinished CAM not remain undisturbed for more than 30 minutes. Spraying of the finished surface with water until application of the curing coat also was required.

Thw two layers of PAM each were compacted with a steel-wheeled roller followed by a vibratory compactor. Special care was required on the part of the operator of the vibratory compactor to prevent shoving and rutting of the material being compacted. The surface of the lower lift was scarified and kept moist until the upper lift was placed. Compaction of each lift to at least maximum density as determined by a somewhat modified version of AASHO T 180 (Method C) was required for each lift and immediately adjacent to the pavement structure, except that a minimum of 97 percent was permitted away from the pavement structure in lifts placed on earth subgrade. The upper lift in each increment of base placed was required to be placed and given its final finish on the same day the lower lift was placed. The surface was to be kept moist until the curing coat was applied.

<u>Curing the CAM and PAM</u>.--The finished base courses of PAM and CAM were sealed with a coat of liquid asphalt (RC-2) which was applied at the rate of 0.15 gal/sq. yd. on the PAM and 0.20 gal./sq. yd. on the CAM. The curing coat was applied on the CAM the same day of the finishing operation, and on the PAM the day following the finishing operation.

<u>Surfacing the CAM and PAM</u>.--The PAM and CAM shoulders were surfaced with a 1 1/2-inch-thick wearing course of bituminous concrete (Class I) placed with a Barber Greene spreader and compacted with rubber-tired and steel-wheeled rollers. The 4-foot median shoulders and the 10-foot outside shoulders were built by the same methods. The compaction operation on a median shoulder is shown in Figure 10.

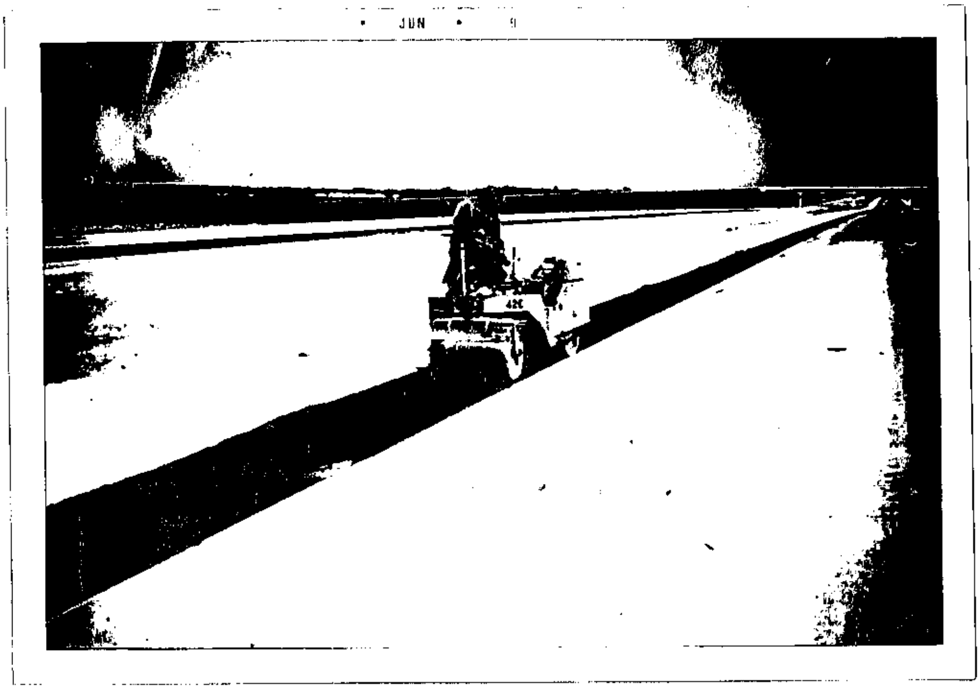


Figure 10. Compacting the bituminous concrete surface on PAM in the median shoulders.

Figure 11. Placing the portland cement concrete shoulders with a slipform paver.

PCC shoulder construction.--The PCC shoulders were placed with a slipform paver pulled by a heavy rubber-tired tractor operating on the pavement (Figure 11). Shoulder concrete was wet-batched from a central mixing plant to the jobsite in agitator trucks. In the finishing operation, dummy-groove transverse joints were hand-troweled 1 1/2 inches deep at 20-foot intervals. The hand-troweled grooves were quite variable in width and depth. After the concrete had hardened, the grooves were re-cut with a saw and filled with the flexible rubber-asphalt joint sealer. Six-foot rumble strips were floated in with a corrugated float at 60-foot intervals. The surface of the concrete was textured with a broom.

No. 5 deformed steel bars, 30 inches long and bent in the middle, were fastened at 30-inch intervals to the mainline pavement reinforcement to be in place for bending outward following the pavement construction to tie the PCC shoulders to the pavement. A cardboard cover was used on the portion of the bars to be bent outward. These bars became dislocated in varying degree during the slipform process that was used to place the mainline pavement concrete, and could be found only with great difficulty after paving. The search for them eventually was abandoned in favor of No. 4 smooth, hooked steel bars, 15 inches long, anchored in the hardened pavement concrete to a depth of 2 inches with expanding end anchors (Figure 12).

TOPSOIL PLACEMENT

Topsoil was placed on the paved shoulders in a windrow, and bladed onto the shoulder berm and slope (Figure 13). Final grading of the shoulder slope re-exposed the subbase on the slope to allow drainage from beneath the shoulder to take place.

JOINT SEALING

A saw cut was made in the pavement-shoulder joint of the west half of each test section to a depth of 3/4-inch and to a width of 1/2-inch, and the cuts

Figure 12. Steel bars were turned into anchors set in the pavement edge to replace lost tiebars.

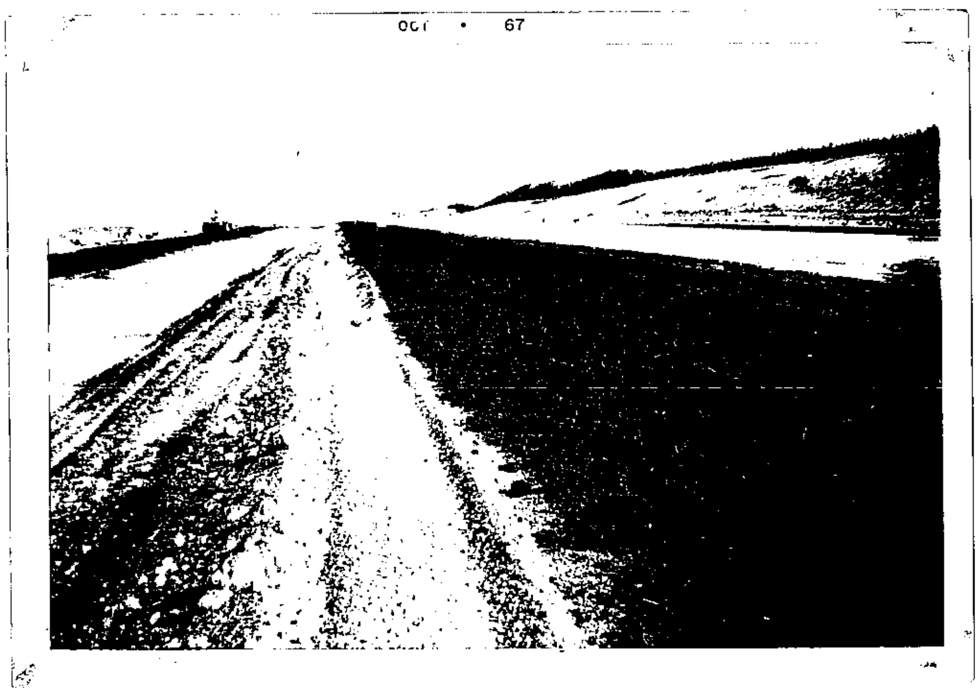


Figure 13. Topsoil windrow on the shoulder was bladed onto the shoulder berm.

sealed with the hot-poured rubber-asphalt sealing compound (Figure 14). No cut was made and no seal was applied in the pavement-shoulder joint in the east half of each test section.

SPECIAL CONSTRUCTION PROBLEMS

During the progress of the shoulder construction work, various problems were encountered that seemed mostly related to the construction materials. Some of the problems are susceptible to correction and can be avoided in future work. Others are inherent in the individual materials involved and do not appear readily susceptible to change.

The washed sand that was furnished to meet the Type A subbase material specification was difficult to compact. It supported the compaction equipment when moistened, but after drying lost its density when disturbed. It was especially troublesome in the median shoulders where it gave way under the 4-foot rollers and allowed them to tip sideways. As mentioned earlier, the Type A material also washed badly during rains, and required reworking before placement of the next structural layers.

Variability in the moisture content of the PAM associated with both weather and water control problems at the mixing plant caused compaction problems on the roadway. The early cutoff date of September 15 for the use of PAM also became a problem on this particular project. The CAM handled well, but difficulties were experienced in obtaining the required density. The BAM tended to roll outward during compaction and ultimately was found to have become consistently thinner than plan thickness.

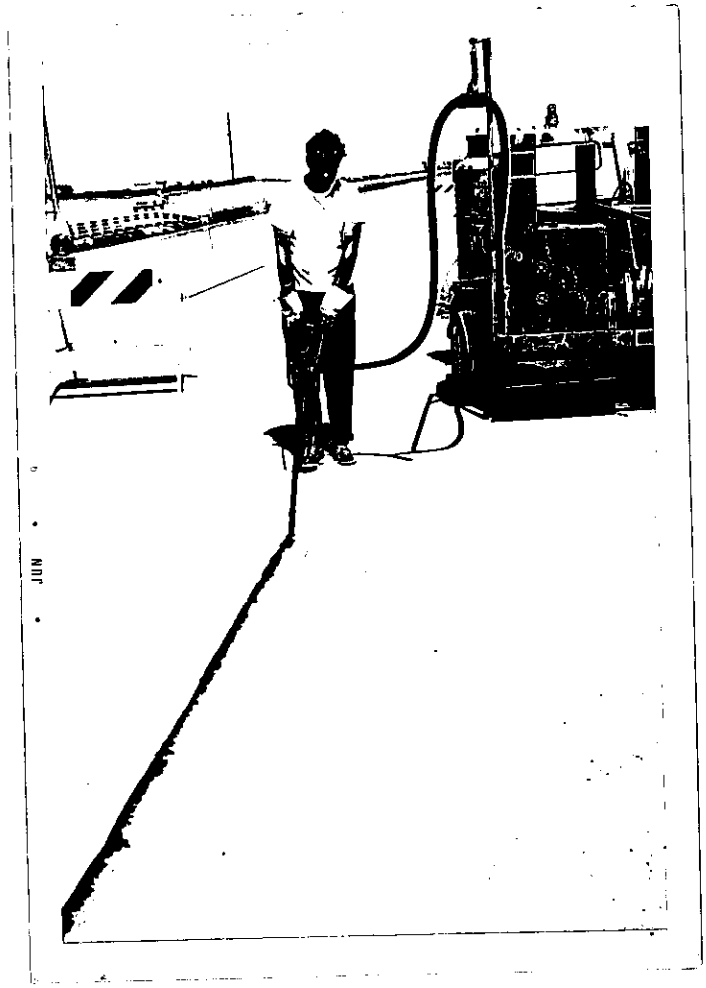


Figure 14. Sawed joints in west half of each test section were sealed with a hot-poured elastic sealant.

SHOULDER COSTS

To obtain some knowledge of the costs of the various shoulder types included in the experiment, analyses were made of (1) unit bid prices and (2) estimated contractor costs. While the results are of interest, it is important to recognize their probable atypical character with reference to normal construction projects.

For estimating contractor costs, the resident engineer on the project maintained daily tabulations showing the pay items on which the contractor was working, the amounts and types of equipment and labor involved in both field and plant operations, the amounts of time that labor and equipment were used for particular pay items, the costs of materials, and the amounts of daily production. The tabulated information was analyzed by estimators in the central office of the Division of Highways, and final cost figures were determined through the application of prevailing wage and equipment rental rates for the area, material costs as quoted by the suppliers, standard overhead costs, and profit factors.

The principal results of the analyses are shown in the bar chart of Figure 15 developed from costs summarized on a per mile basis for shoulders adjacent to one pavement. It is emphasized that the per mile costs that are shown for the various shoulder designs and types must be recognized as applying to experimental construction, and in some instances to types of construction unfamiliar to the contractor. Under these unique conditions, the cost figures and cost relationships that are indicated may not be truly representative of conditions that might exist in normal situations.

Insofar as the contract costs to the State are concerned, it will be noticed that less than $5,000 per mile separates the four shoulder types, with the PCC shoulders costing the most and the PAM shoulders costing the least. This is true for both the shoulders with subbase and those without subbase. The use of subbase will be seen to have added a little over $7,000 per mile to the contract prices.

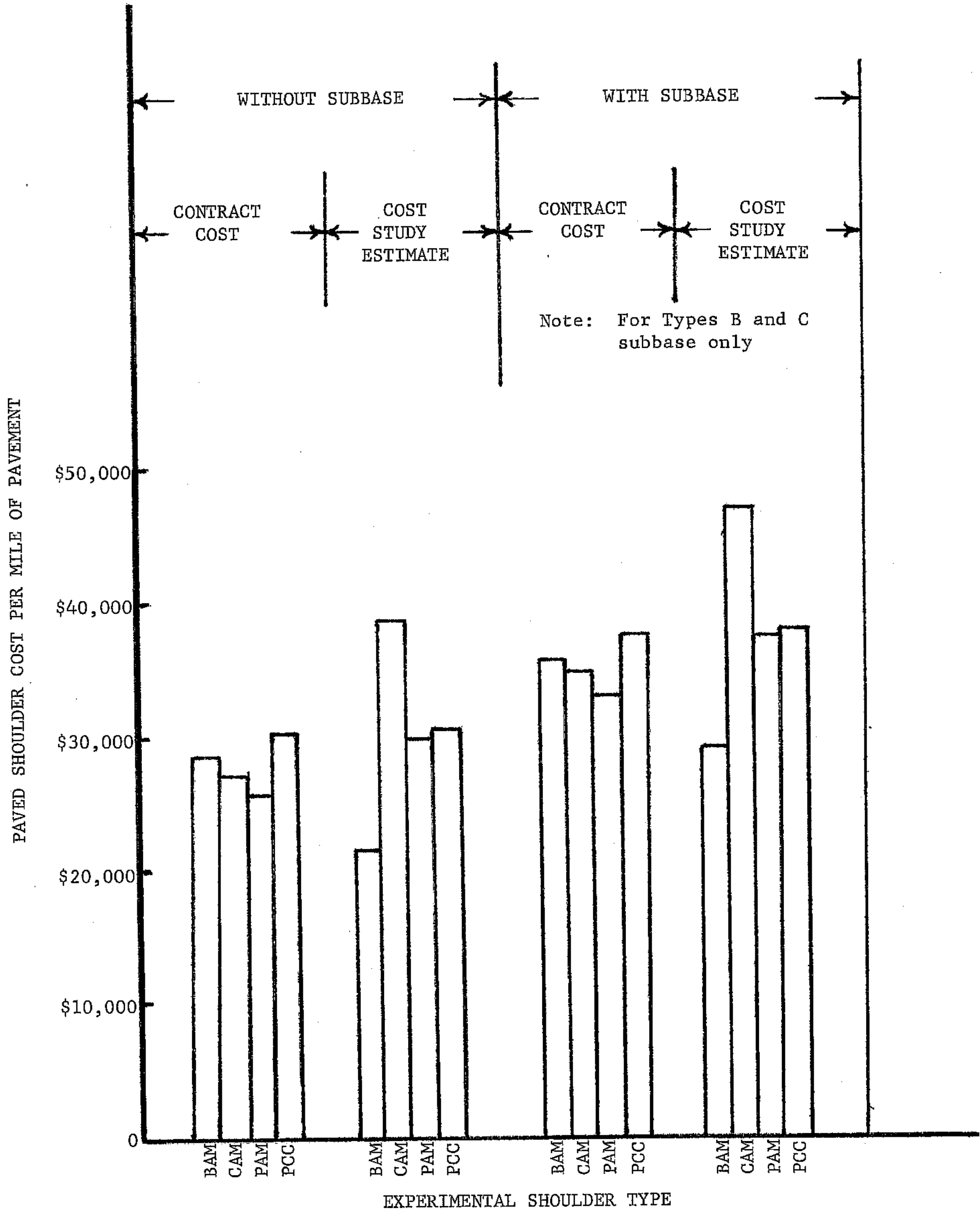


Figure 15. Shoulder construction costs on experimental project

For this single project, the cost study figures favor BAM construction. The cost that is indicated for the CAM construction, which is considerably higher than the cost of constructing any of the other types, seems singularly high and at variance with the situation that existed in Illinois a few years ago when alternate bids were being invited on CAM and BAM shoulders.

It will be noted also in the figure that the use of Types B and C (coarse aggregate) subbases adds about $7,000 to $8,000 per mile to the computed construction costs. The cost of the Type A (sand) subbase that proved to be impractical because of construction problems was higher than that for the Types B and C subbases, and was not included in the development of the chart.

RESEARCH MEASUREMENTS DURING CONSTRUCTION

A research observer was present on the site continuously during construction of the pavement and shoulders, beginning with the fine grading of the pavement subgrade and ending with the placement of the topsoil on the shoulder berms. Notes were made on all construction operations and material usage. Quality control sampling and testing procedures were observed, and installation of the research instrumentation was supervised.

Dry densities and moisture contents of the upper 6 inches of embankment soil were determined at 300-foot intervals in both pavement and shoulder areas immediately prior to placement of the next succeeding structural layer. Measurements of moisture and density also were made at numerous locations in the shoulder base course materials following compaction. Nuclear testing equipment was used for these measurements. A summary of the data that were obtained in the subgrade soil measurements is shown in Table 4 presented earlier in the report.

POST-CONSTRUCTION MEASUREMENTS

During the post-construction period, an effort was made to develop some new knowledge on the influences that the general environment might have on shoulder behavior and on shoulder movement. Weather records were examined, frost penetration was observed, soil moisture contents and densities were monitored, and elevation changes in the pavement and shoulder components and in the underlying soils were checked. These studies were in addition to condition surveys and an extensive coring program to examine the durability of the shoulder base and the pavement subbase mixtures. The locations principally involved in the post-construction measurements program are shown in Table 5.

Weather Conditions

Average monthly maximum and minimum temperatures compiled at the U. S. Weather Station at Joliet during the study period, and the same data compiled by the station at Chicago Midway Airport near the Stevenson Expressway in 1964-65, were used in compiling the graphs of Figure 16. Precipitation data from the same stations were used in preparing the bar charts of Figure 17. Additional summary data are presented in Table 6.

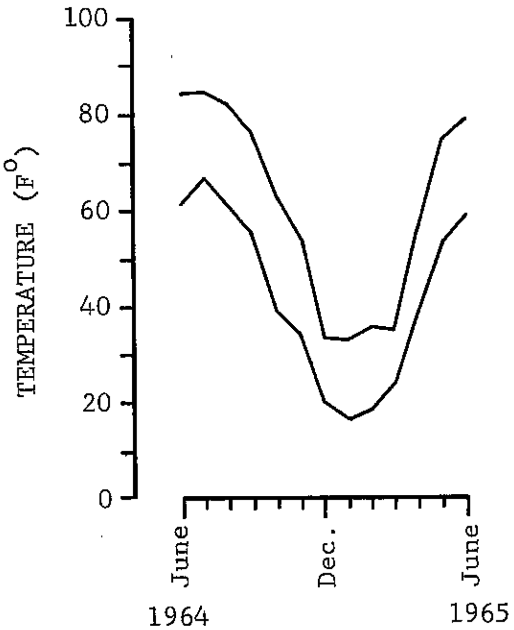
The mean annual precipitation at Joliet is about 33 inches, with about 32 inches of snowfall. On the average, there are about 9 days per year with icy conditions occurring some time during the day (4). There are about 12 days per year with subzero temperatures and 28 with temperature exceeding 90F.

Cumulative freezing degree-day curves were charted in Figure 18 for each freezing season to establish the Freezing Indices shown in Table 6. Each degree in any one day in which the mean between the maximum and minimum temperatures for that day varies below freezing is called a degree day. The Freezing Index is the

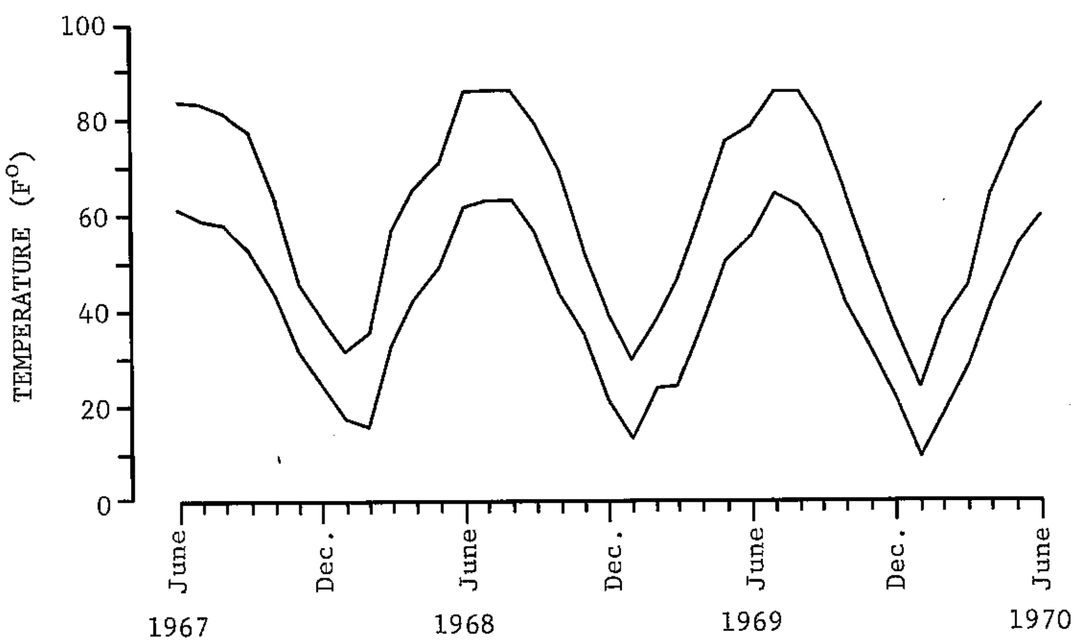
TABLE 5

LOCATIONS INVOLVED IN POST-CONSTRUCTION MEASUREMENTS PROGRAM

Station	Shoulder Type	Shoulder Subbase	Joint Sealed	Settlement Plates		Frost Gage		Moisture-Density
				Pavement	Shoulder	Pavement	Shoulder	
				Eastbound Roadway				
462+50	CAM	Yes	Yes	X	X			X
464+50	"	"	No	X	X			X
487+00	BAM	"	Yes	X	X			X
495+00	"	"	No	X	X	X	X	X
537+50	CAM	"	Yes	X	X			X
542+00	"	"	No	X	X			X
548+00	PAM	"	Yes	X	X	X	X	X
553+50	"	"	No	X	X			X
619+00	PCC	"	Yes	X	X			X
626+00	"	"	No	X	X	X	X	X
647+00	BAM	"	Yes	X	X			X
649+00	"	"	No	X	X			X
				Westbound Roadway				
462+50	CAM	No	Yes	X	X			X
464+50	"	"	No	X	X			X
487+00	BAM	"	Yes	X	X			
495+00	"	"	No	X	X		X	
537+50	CAM	"	Yes	X	X			X
542+00	"	"	No	X	X			X
548+00	PAM	"	Yes	X	X		X	X
553+50	"	"	No	X	X			X
619+00	PCC	"	Yes	X	X			X
626+00	"	"	No	X	X		X	X
647+00	PAM	"	Yes	X	X			X
649+50	"	"	No	X	X			X

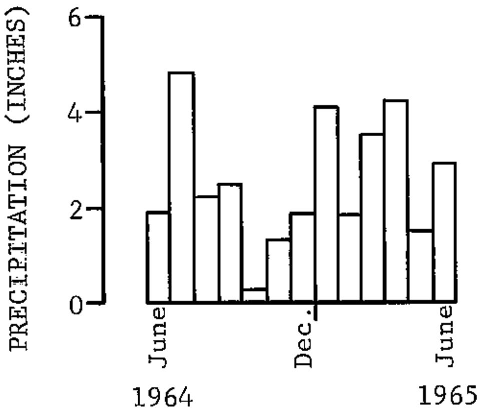


MIDWAY AIRPORT WEATHER STATION

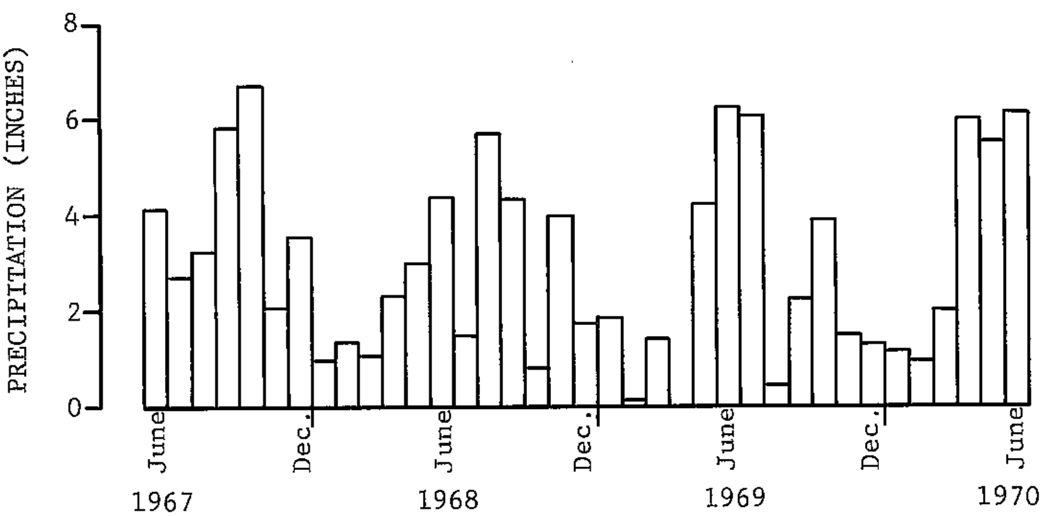


JOLIET WEATHER STATION

Figure 16. Average monthly minimum and maximum temperatures at Midway Airport and at Joliet, Illinois.



MIDWAY AIRPORT WEATHER STATION



JOLIET WEATHER STATION

Figure 17. Average monthly precipitation at Midway Airport and Joliet, Illinois.

TABLE 6.

WEATHER SUMMARY

Characteristics		Midway Airport 1964-65	Joliet 1967-68	Joliet 1968-69	Joliet 1969-70
Mean Winter Temperature, F		26.4	27.0	26.8	24.2
Precipitation, in.	(Summer)	14.3	11.5	12.7	11.4
	(Fall)	8.4	9.0	7.6	15.8
	(Winter)	7.7	5.7	3.6	3.3
	(Spring)	9.1	6.4	9.9	13.6
Snowfall, in.		59.6	14.1	11.0	45.0
Freezing index, degree days		654	573	515	713

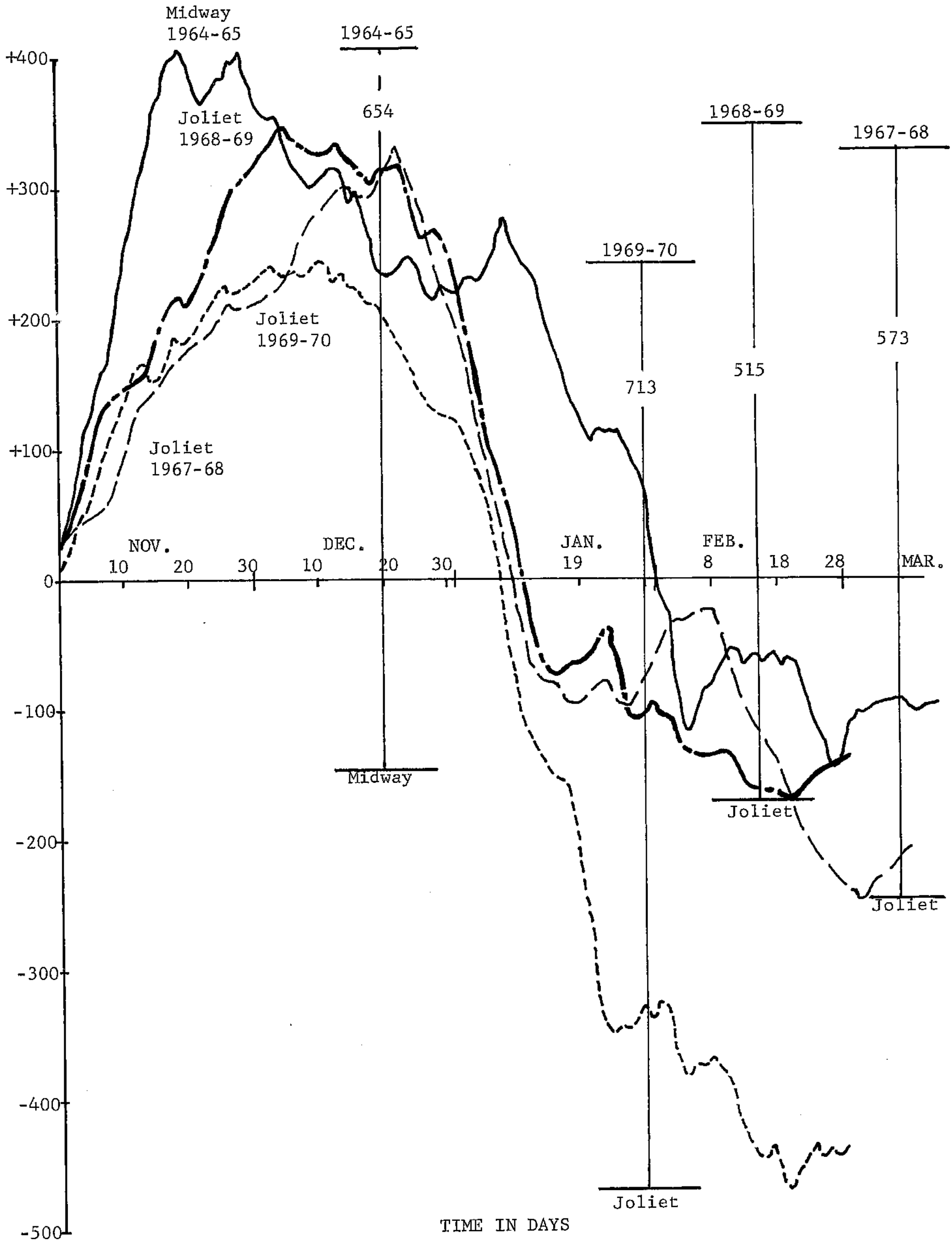


Figure 18. Freezing degree-day curves for Joliet, Illinois and Midway Airport.

number of degree days between the highest and lowest points on the degree-day time curve for one freezing season.

Differences between the climatological records of the Midway Airport station in the 1964-65 period during and immediately following construction of the Stevenson Expressway and of the Joliet station in 1967-68 during and immediately following the construction of the experimental shoulder section on Interstate 80 do not appear to be of a magnitude that would suggest responsibility for the wide differences in behavior of the shoulders at the two locations. The very early onset of freezing weather in late 1964 as indicated by the freezing degree-day curve on Figure 18, which coincided with the early observance of shoulder heaving on the Stevenson Expressway, is perhaps worth noting.

From the data on hand, only general qualitative relationships between the pavement and shoulder movements and the climatic environment are observable. More discussion will be offered on this subject later in the report.

Frost Penetration

Frost depth was monitored principally with resistance-type gages in which the detection terminals were spaced at one-inch intervals to a depth of 48 inches below the surface (5). The presence of frost is indicated by a major change in soil resistance at frost depth. Resistance measurements usually were made by hand with an AC bridge, but also were monitored on occasion at one location at two-hour intervals with a Bristol multi-channel recorder. Resistance-type gages were installed under the eastbound and westbound outer shoulders and under the eastbound pavement at each of three station locations that coincided with station locations of the settlement plates. A total of nine of the gages were installed. A sketch of a frost gage installation is shown in Figure 19.

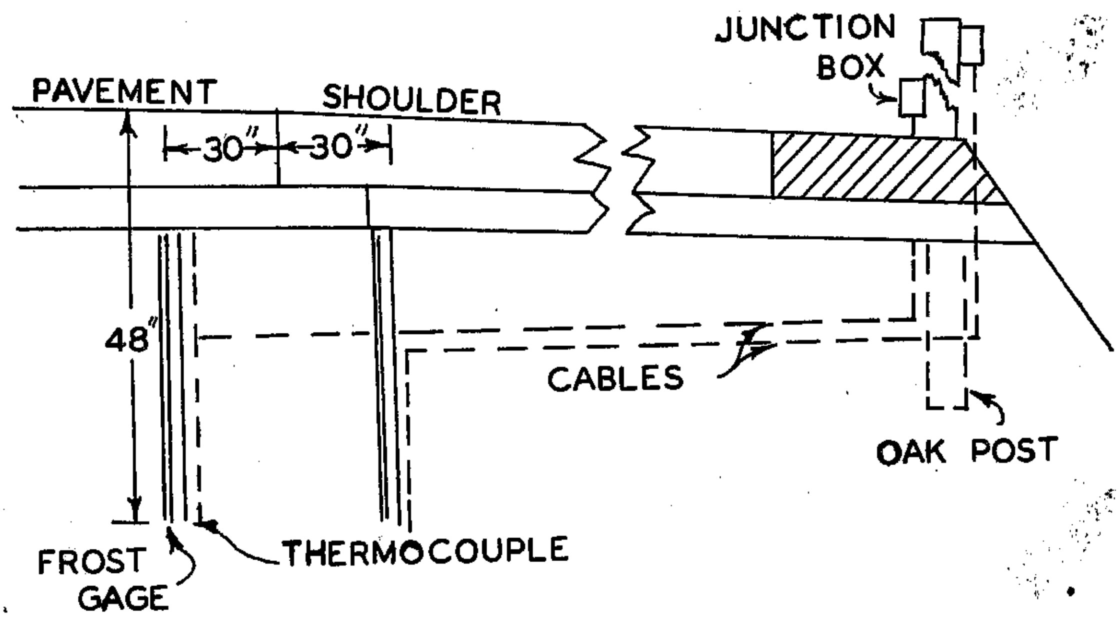


Figure 19. Frost gage installation.

Maximum frost depths that were recorded on the frost gages are shown in Table 7. It will be noted that during the three winters that the records were maintained the maximum depth readings at the individual sites for the most part did not vary greatly from year to year; nor did the maximum depths of penetration vary much between the pavement and adjacent shoulders. The greatest variations occurred between site locations. Frost penetrated below subgrade level at all measurement locations at some time each winter season during the period of study.

Examination of Figure 18 shows that there were two distinct freezing cycles during the winter of 1967-68 at Joliet. In the period between cycles, the frost disappeared completely from beneath the pavement and shoulders of the experimental sections. The first freeze cycle ended about the middle of January and produced 427 freezing degree-days. The second freeze cycle began during the second week in February and produced an additional 220 freezing degree-days. In the winters of 1968-69 and 1969-70, the soils remained permanently frozen except that some thawing occurred at the soil surface during short warm periods. During the winter of 1967-68, a count was made from the frost gage data of the number of times the frost disappeared from the soil at depths of 12, 18, and 24 inches below the pavement or shoulder surface at the frost monitoring stations. Count results are shown in Table 8. As would be expected, the table shows that the soil thawed more times at 12 inches below the surface than at 18- and 24-inch depths. In the winters of 1968-69 and 1969-70, there were no thaws at 18 and 24 inches below the surface.

As a side experiment to determine the accuracy of various types of frost-depth gages, temperature gages were installed beside all of the resistance-type gages, and Gandahl gages (6) were installed at two of the locations. The temperature gages consisted of copper-constantan thermocouples installed at 4-inch intervals an inch away from each of the resistance-type gages. Temperatures were measured

TABLE 7

MAXIMUM DEPTHS OF FROST PENETRATION UNDER PAVEMENT AND SHOULDERS OF I-80

Year	Station	Direction	Freezing Index (freezing deg. days)	Maximum Frost Depth Pavement (in.)	Maximum Frost Depth Shoulders (in.)
1967-68	495+00	EB	573	27	28
		WB		-	30
	548+00	EB		28	26
		WB		-	27
	626+00	EB		34	36
		WB		-	32
Average				30	30
1968-69	495+00	EB	515	30	14
		WB		-	32
	548+00	EB		24	25
		WB		-	28
	626+00	EB		33	34
		WB		-	34
Average				29	28
1969-70	495+00	EB	713	29	29
		WB		-	44
	548+00	EB		24	25
		WB		-	28
	626+00	EB		37	37
		WB		-	40
Average				30	34

TABLE 8

NUMBER OF FREEZE-THAW CYCLES BY LOCATION
AND DEPTH BELOW THE PAVEMENT SURFACE
1967-68

Station	Design 1/	Depth Below Pavement Surface (in.) 12	18	24
495+00				
EB Pavement	PCC/BAM	3+	1	1
EB Shoulder	BAM/B	3+	1	1
WB Shoulder	BAM/E	2	1	1
548+00				
EB Pavement	PCC/PAM	2	2	1
EB Shoulder	PAM/B	2	1	1
WB Shoulder	PAM/E	2	2	1
626+00				
EB Pavement	PCC/CAM	2	2	1
EB Shoulder	PCC/B	2	2	1
WB Shoulder	PCC/E	2	2	2

1/ E of PCC/E refers to earth subgrade; B refers to Type B subbase.

with a Leeds and Northrup potentiometer calibrated to read degrees F. Freezing was assumed to take place in the soil at 31.8F. The Gandahl gage is a tube containing liquid calibrated to read depth below the surface. Frost depth is determined by removing the tube from the ground and determining the depth below the surface to which the liquid column is frozen.

The average apparent frost depths indicated during January and February 1969 by each of the three types of gages at the two observation stations where all were installed are shown in Figure 20. Mean daily air temperatures for the same period at the Joliet weather station are also plotted in the figure. Apparent frost depths indicated by all three methods of measurement agreed closely during the first part of the period but diverged during the latter part. The reason for the divergence is not entirely clear. Auger borings made through the frozen soil under the shoulder indicated that the resistance gages were measuring the frost depth most precisely.

Frost penetration will be discussed further in connection with the settlement plate readings.

Moisture-Density

Soil densities and moisture contents in the shoulder area subsequent to construction were measured with a nuclear probe through two-inch diameter wells drilled in the shoulders. The measurement procedure was a variation of a method used by Marks and Haliburton in Oklahoma (7). The moisture-density measurements were made at most of the shoulder locations (see Table 5) where instrumentation was installed for the measurement of vertical movements. The measurements were made first in May 1969 and repeated in October 1969, and in May and October 1970. Summary results of the measurements are shown in Table 9. Results of moisture-density measurements

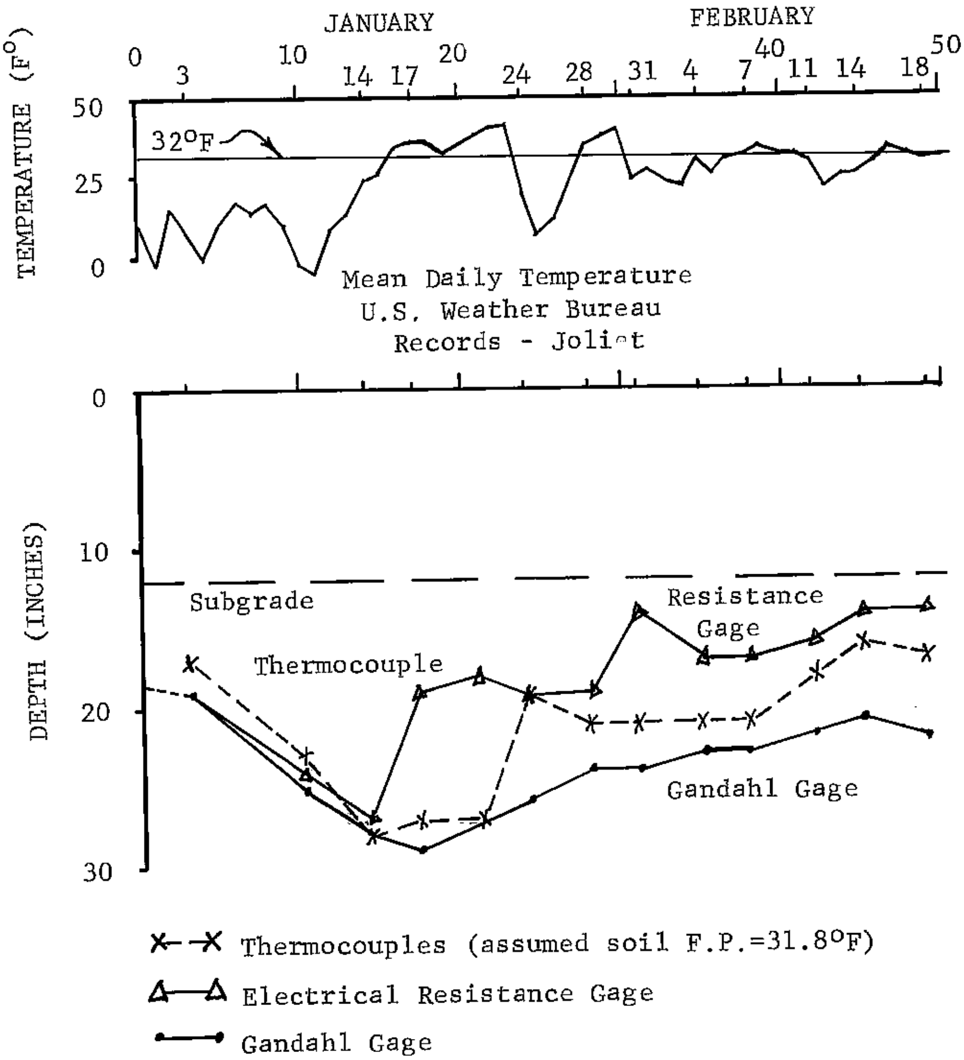


Figure 20. Comparison of methods of frost depth measurement.

TABLE 9

AVERAGE MOISTURE CONTENTS AND DENSITIES OF SUBGRADE SOILS IN SHOULDER AREA

			1967		1969				1970			
			At Construction		May		October		May		October	
Shoulder Subbase	Joint Seal	Depth below Subgrade (in.)	Moisture Content (%)	Density (pcf)	Moisture Content (%)	Density (pcf)	Moisture Content (%)	Density (pcf)	Moisture Content (%)	Density (pcf)	Moisture Content (%)	Density (pcf)
Yes	Yes	1			16.4	109.3	17.8	108.0	16.8	112.9	18.9	106.7
Yes	Yes	2			16.8	111.5	17.5	110.9	16.7	115.9	20.0	105.7
Yes	Yes	4			17.3	113.9	17.2	113.2	17.6	115.5	19.6	106.2
Yes	Yes	6	9.9	116.4	17.4	115.3	16.9	115.2	17.3	118.1	19.0	107.4
Yes	Yes	12			18.9	107.1	18.4	106.5	18.0	114.6	20.2	106.0
Yes	Yes	24			18.6	114.5	17.3	115.1	17.5	118.0	19.6	106.6
Yes	Yes	36			18.3	112.9	17.2	113.7	18.4	113.2	18.5	105.5
Yes	No	1			18.8	111.1	17.6	109.8	18.4	110.2	20.5	105.8
Yes	No	2			19.1	113.4	17.6	114.1	18.7	112.9	20.6	106.1
Yes	No	4			18.4	118.3	17.1	118.4	18.2	117.6	20.7	105.5
Yes	No	6	10.0	113.3	18.2	118.4	17.4	117.5	18.6	116.1	20.7	106.1
Yes	No	12			19.6	115.3	18.6	114.5	18.4	117.1	20.9	105.7
Yes	No	24			19.5	113.9	18.7	113.4	19.1	115.2	21.3	105.7
Yes	No	36			20.2	111.8	19.4	110.7	20.1	113.5	20.8	106.3
No	Yes	1			18.8	107.4	20.6	104.6	19.8	104.1	20.6	105.8
No	Yes	2			18.3	109.6	20.0	106.3	21.0	101.2	20.1	106.4
No	Yes	4			17.2	115.8	18.8	111.4	18.8	112.1	19.8	108.5
No	Yes	6	8.0	114.1	17.3	114.7	18.5	111.6	18.2	114.0	18.7	109.9
No	Yes	12			17.8	113.3	18.8	110.4	18.6	113.0	19.6	107.9
No	Yes	24			18.1	113.3	18.6	110.3	19.2	109.9	19.4	108.6
No	Yes	36			17.9	112.3	19.3	109.1	19.8	106.6	19.0	111.9
No	No	1			18.6	108.5	17.4	107.6	19.7	110.1	19.3	107.8
No	No	2			17.9	110.6	16.7	109.8	19.1	110.7	18.6	109.9
No	No	4			16.8	115.0	15.6	114.3	17.4	112.6	17.7	110.4
No	No	6	8.4	113.2	17.5	113.6	16.5	112.9	18.0	111.2	18.8	108.2
No	No	12			18.6	110.8	17.7	111.0	19.2	111.1	19.2	111.4
No	No	24			17.4	112.9	15.6	112.3	18.0	111.9	17.0	114.0
No	No	36			17.8	115.2	17.2	114.3	19.9	112.7	19.6	110.9

1/ Moisture percentages and density values for 1967 were obtained with nuclear gages on the surface and represent the moisture and density condition in the upper 6-inch soil layer.

Average Plastic Limit = 18.3 percent.

made at subgrade level immediately prior to placing the next construction course are also shown for comparison.

While some inconsistencies are obvious in the data of Table 9 taken from the nuclear probe readings, it is believed that the information is sufficiently accurate for some general observations of apparent trends. For example, a substantial moisture gain subsequent to construction is evident. Also, a comparison of the 1969 and 1970 moisture content data of Table 9 with plastic limit data of Table 2 and the optimum moisture content data of Table 4 shows that moisture contents following construction are generally near the plastic limits and somewhat above the optimum moisture contents. No strong trends with respect to field densities are distinguishable, except that higher densities seem often to be associated with lower moisture contents. The possibility of the existence of a slight trend regarding the association of higher moisture levels with unsealed shoulder-pavement joints can be suspected from the data. No strong trend that would associate soil moisture level with the presence or absence of the open-graded aggregate subbase is observable in the data.

The moisture-density relationships that exist in subgrade soils beneath pavements and paved shoulders are undoubtedly extremely complex. The amount of moisture present in the soils, in addition to being related to the physical characteristics and initial void contents of the soils, is related to the availability of groundwater, the amount of precipitation and related runoff, and to the perviousness of the structural members. The amount and condition of the cracks in pavement and shoulder, the treatment of joints, the permeability of subbase materials, the rapidity with which water is removed from subbase materials, and the configuration of the surface with respect to the flow of surface water from the pavements and shoulders are therefore all factors of influence.

In view of all the complexities that prevail, it is not surprising that the moisture-density study furnished only qualitative information.

Movements of Pavements and Shoulders

Settlement plates for measuring elevation changes were installed under the pavements and outside shoulders in the eastbound and westbound roadways at 12 station locations, for a total of 48 separate pavement-shoulder installations. The number of plates differed between installations, depending on the number of pavement or shoulder structural components. A typical settlement plate installation in a shoulder with a subbase is shown in Figure 21. A steel rod passes downward from the pavement or shoulder surface to each plate, passing through a steel casing pipe which is plugged at the surface to keep out dirt and water. Elevations were determined with an engineering level and rod, and referenced to six permanent benchmarks installed so that settlement plate locations were, in no instance, more than 500 feet distant.

Observing the change in elevation of the settlement plates permits a study of vertical movements of both the pavement and the shoulders and, at the same time, changes in the thickness of the structural components. Installation sites were situated such that each shoulder type could be studied both on a Type B crushed-stone subbase and on the earth subgrade, with and without sealed pavement-shoulder joints. Winter elevations of each settlement plate were determined monthly, except during the last winter (1969-70) when only one set of observations was made. Readings were also made once during the spring, summer, and fall seasons (April, July, November) of each year.

Plots of the changes in surface elevation of pavement-shoulder pairs, using the original elevation as zero, show the total vertical movement of the pavement and the shoulder at each site and permit observation of the differences in movement

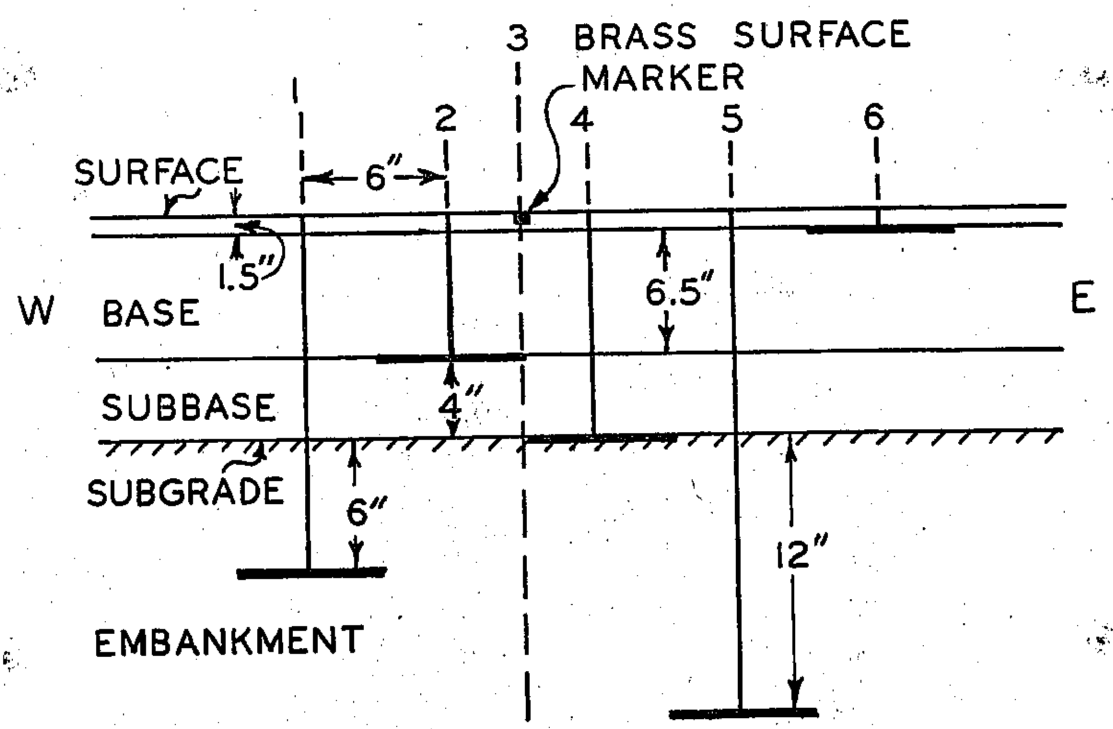


Figure 21. Settlement plate installation.

between pavement and shoulder at the same time. The surface elevations of each pavement-shoulder pair are plotted for all the observation dates arranged by shoulder type in Figures 22, 23, 24, and 25. The vertical axis shows the change in elevation of the pavement and the shoulder on each observation date referenced to the original elevation as zero. The difference between the pavement curve and the shoulder curve of each pair represents the total differential movement that has occurred between pavement and shoulder since the date of the first measurement.

It is evident from the figures that the pavements and shoulders rose during each freezing season and settled when the frost disappeared. The magnitudes of the movements were irregular and unique to the individual sites. The frost rise amounted to as much as 0.14 feet but was more ordinarily in the order of half that amount or less. Frost heaves were smaller during the first winter after construction than during the second and third winters. Maximum frost heaves occurred during the coldest periods of the year. In the frost-free season, the pavements and shoulders settled back to an elevation which was not necessarily the original elevation. Final elevations were above, below, or approximately the same as the original elevation.

Typically, the direction of movement of the pavements and associated shoulders, both upward and downward, has been the same during any given time period. More often than not, the rise of the shoulders has been slightly greater than that of the pavements, but this pattern has not been completely consistent. The differential movements between shoulders and adjoining pavements have been much smaller than the movements of both. No consistent trend that can be attributed to a particular shoulder type, other than the absence of differential movement where the PCC shoulders were tied to the adjacent pavements, is observable. No general shoulder upheavals

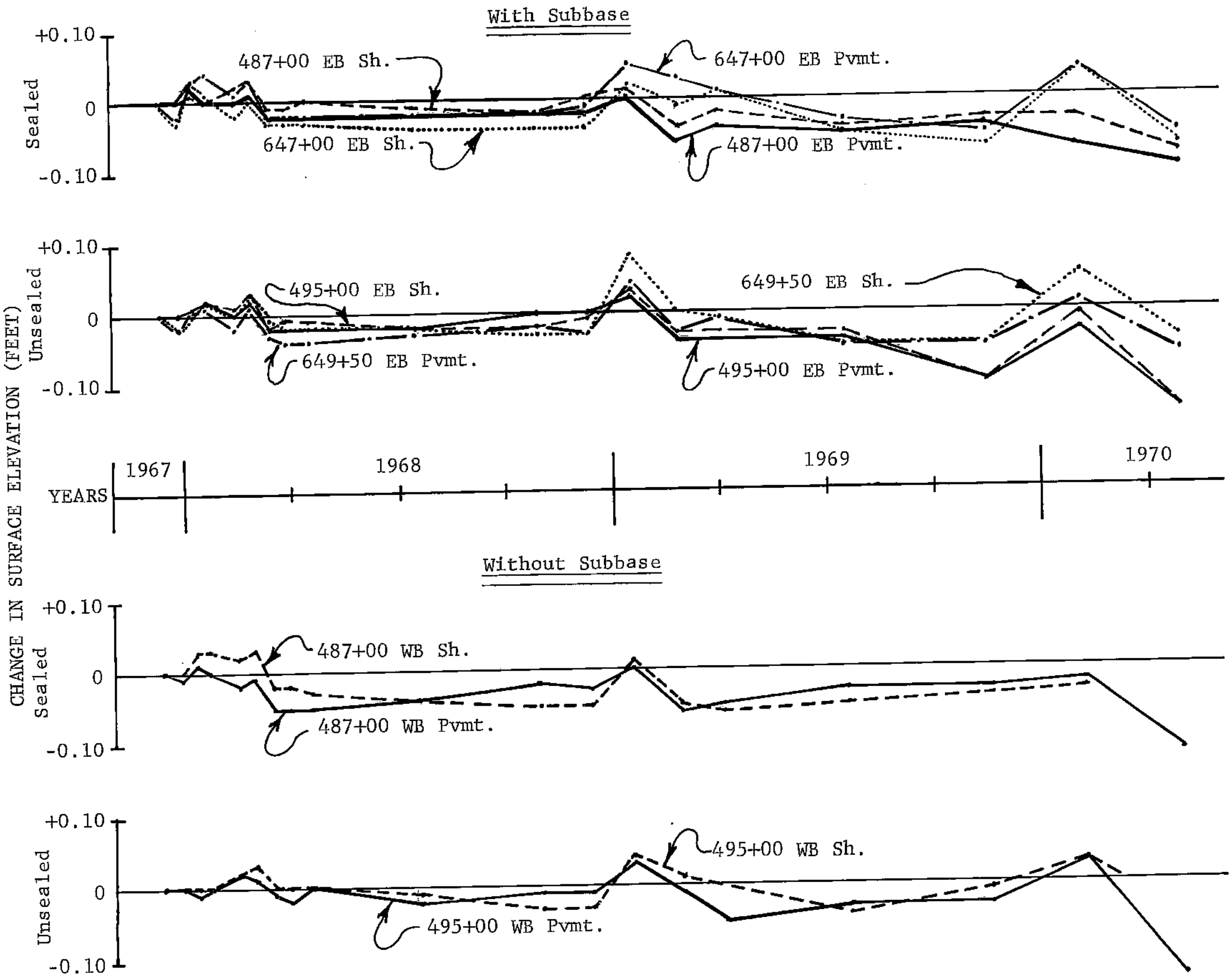


Figure 22. Differential surface elevation change in BAM shoulders.

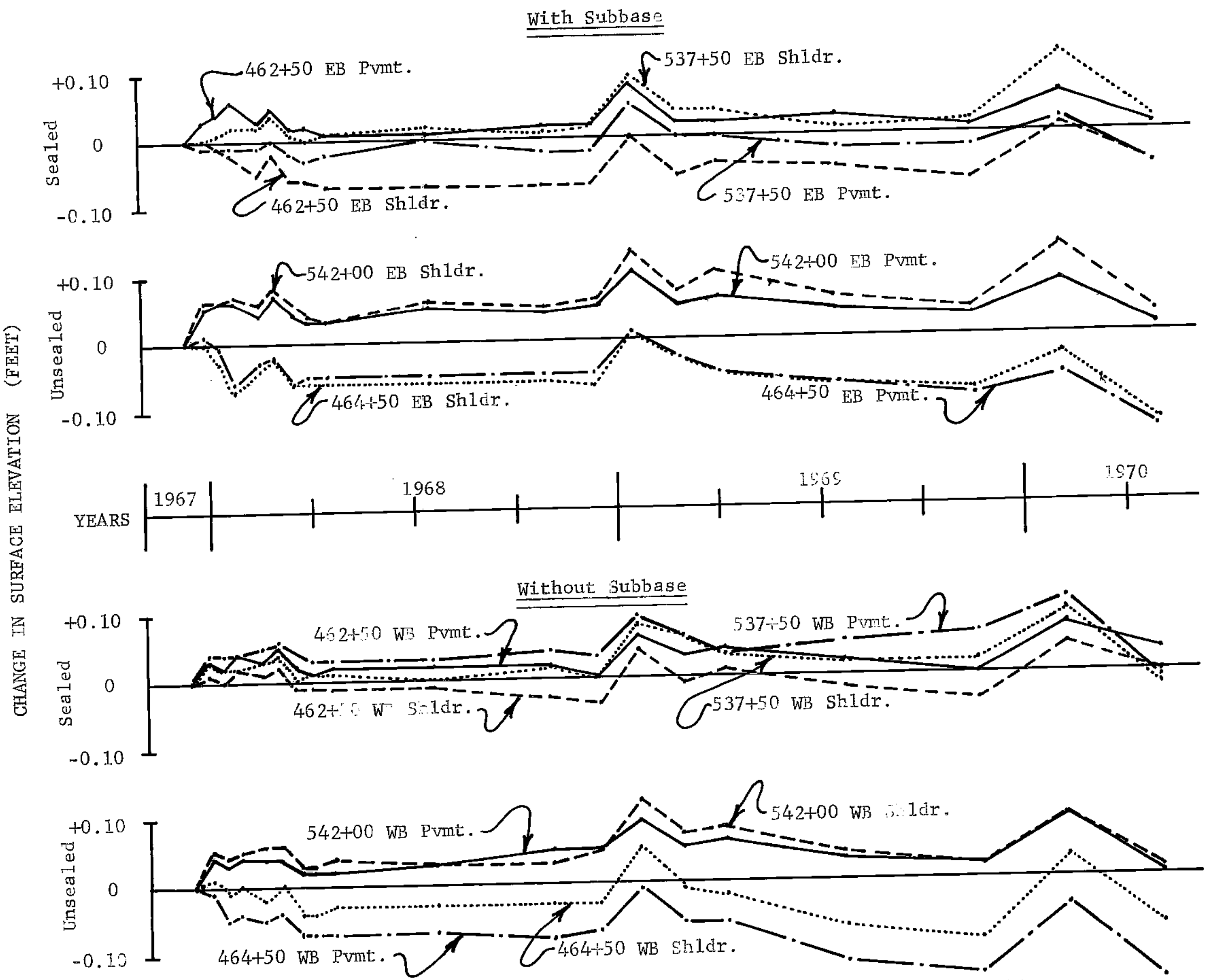


Figure 23. Differential surface elevation change in CAM shoulders.

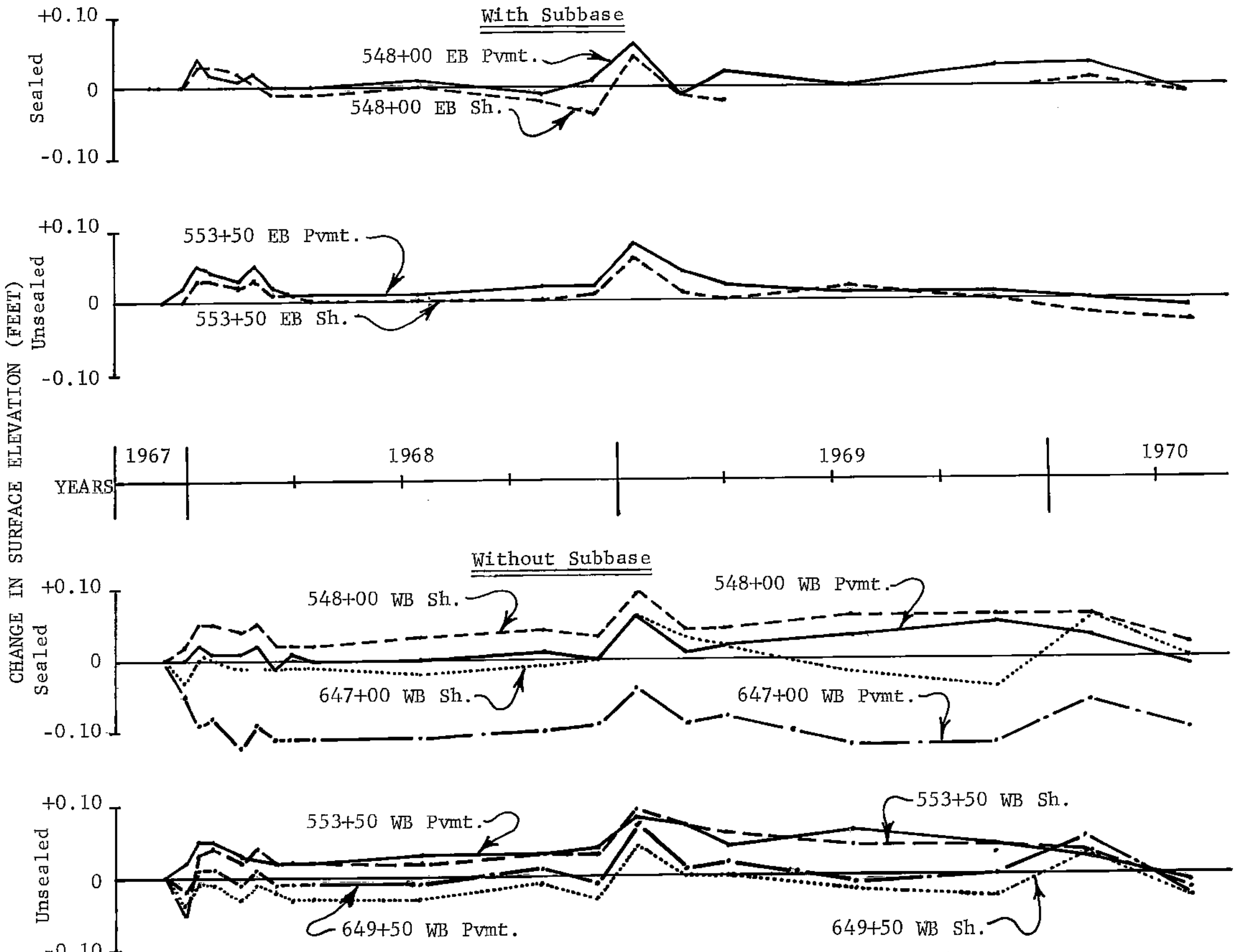


Figure 24. Differential surface elevation change in PAM shoulders.

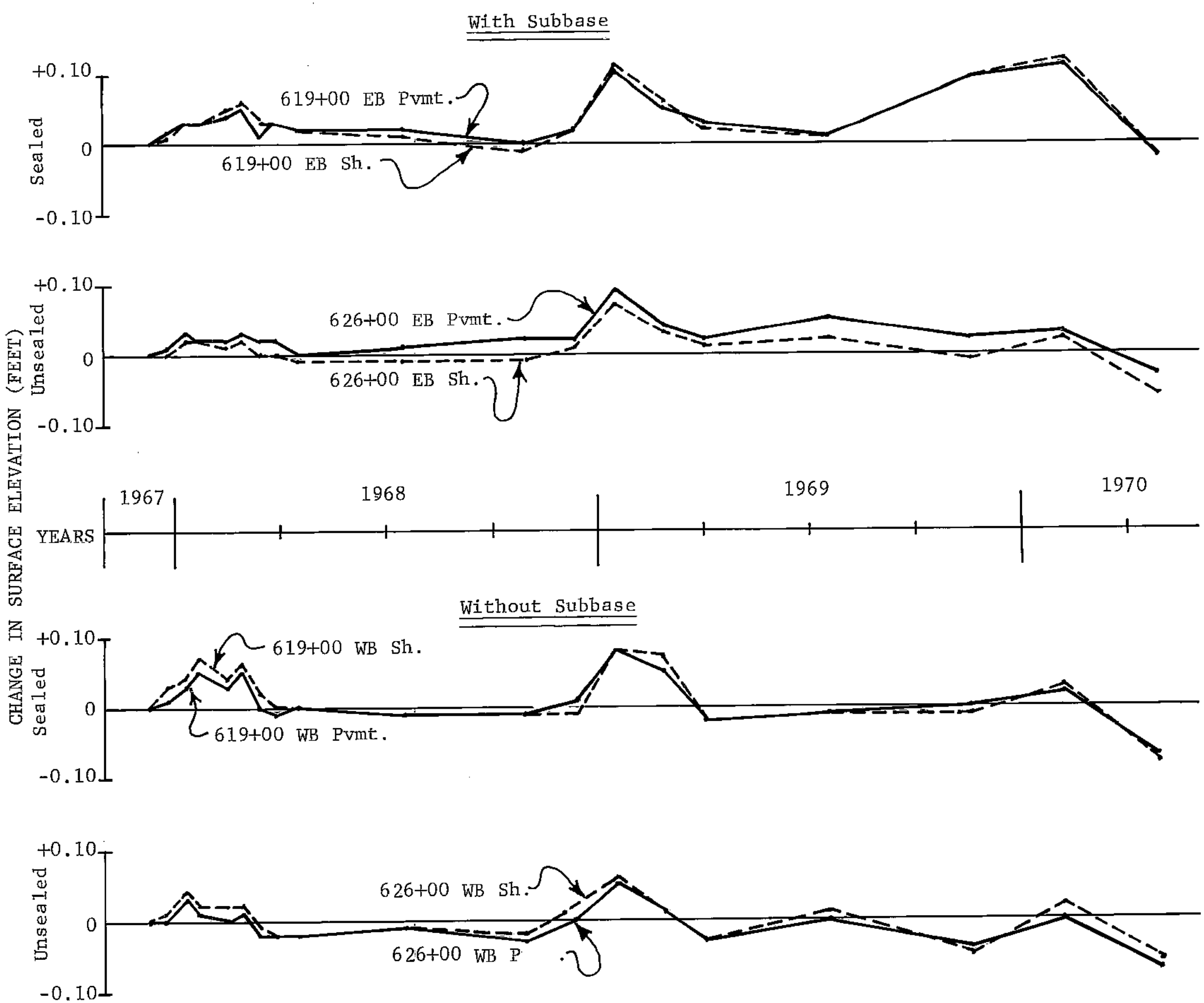


Figure 25. Differential surface elevation change in PCC shoulders.

of the magnitude of those that took place on the Stevenson Expressway during the winter of 1964-65 have occurred. This difference is not explainable through a comparison of weather records for the periods.

To investigate whatever relationships might exist between winter upward surface movements and changes in thicknesses of the pavement and shoulder components and in the subgrade soils, the settlement plate elevations obtained just prior to each freezing season were compared with the elevations taken at the time of maximum frost heave. The comparison was made for the last two winters of study (1968-69 and 1969-70) and the results combined to produce the averages shown in Table 10. Although some inconsistencies are obvious, a trend toward an accounting for practically all of the upward movement within the foundation soil, and a significant portion within the upper 12 inches of foundation soil, is evident. No other trends are observable in the data.

Lateral movements of the shoulders away from the pavements have been minor, but sufficient to produce an opening through which surface water can penetrate readily in the unsealed-joint sections. This does not apply to the concrete shoulders where the tiebars have kept the pavement-shoulder joint tightly closed.

To gain some insight regarding possible relationships between the climatologic environment, frost penetration, and winter heave of the pavements and shoulders, Table 11 and Figure 26 were prepared.

It was pointed out earlier (in connection with Table 7) that, for the period of study, maximum depths of frost penetration varied to some extent from year to year and between adjacent pavement and shoulder pairs, and in greater amount between the individual site locations. Evidently, environmental conditions, undoubtedly including microclimate, unique to each site had a substantial effect.

Average maximum frost penetration data, winter heave data, and freezing index data are presented in Table 11. A general relationship between frost depth and

TABLE 10

SUMMARY OF THICKNESS CHANGE IN PAVEMENT AND SHOULDERS
ASSOCIATED WITH FREEZING AT JOLIET

			Thickness Change (Feet)							
			Pavement				Shoulder			
					Soil				Soil	
Shoulder	Shoulder Subbase	Sealed Joint	Surface Elev.	Subbase	0-12"	12"+	Surface Elev.	Subbase	0-12"	12"+
BAM	Yes	No	+.05	0	+.01	+.04	+.07	-	+.04	+.03
	No	No	+.05	0	+.01	+.03	+.05	-	+.01	+.03
	Yes	Yes	-.01	+.01	-.01	0	+.01	0	+.04	-.03
	No	Yes	+.02	-.04	+.04	-.01	+.04	-	+.04	0
CAM	Yes	No	+.05	0	+.05	+.01	+.08	+.01	+.04	+.03
	No	No	+.06	+.02	0	+.04	+.07	-	+.02	+.03
	Yes	Yes	+.06	+.01	+.04	+.01	+.09	0	+.06	+.02
	No	Yes	+.06	-.02	+.01	+.03	+.08	-	+.06	+.01
PAM	Yes	No	-.03	0	+.01	+.01	+.03	0	+.08	-.04
	No	No	+.01	0	+.04	-.03	+.03	-	+.02	-.01
	Yes	Yes	+.03	0	+.02	+.03	+.04	-	+.04	+.02
	No	Yes	+.02	+.01	+.06	-.04	+.03	-	+.01	-.02
PCC	Yes	No	+.03	0	+.05	+.01	+.05	0	+.02	+.03
	No	No	+.05	0	+.04	+.01	+.05	-	+.05	+.01
	Yes	Yes	+.10	-.05	+.08	+.02	+.12	-.01	+.04	+.03
	No	Yes	+.05	+.01	+.03	+.02	+.07	-	+.03	+.04

TABLE 11

FROST DEPTH AND WINTER HEAVE RELATED TO FREEZING INDEX

Date	Freezing Index (degree-days)	Maximum Frost Depth (inches)		Maximum Winter Heave (feet)	
		Pavement	Shoulder	Pavement	Shoulder
1967-68	573	29.7	29.8	.033	.028
1968-69	515	29.0	27.8	.050	.055
1969-70	713	30.0	33.8	.054	.066

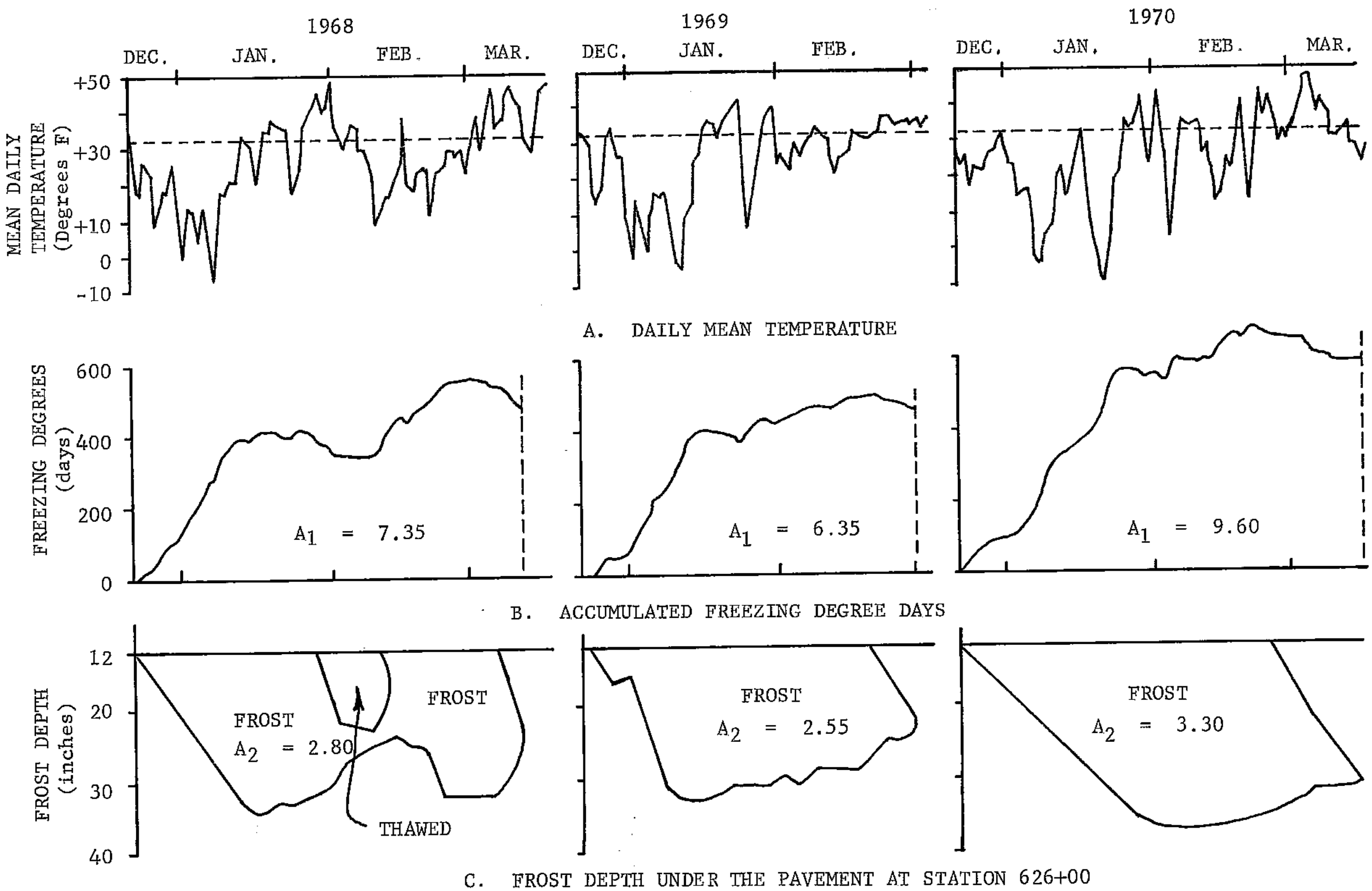


Figure 26. Relation between Freezing Degree Days and Frost in the Subgrade Soils at Joliet

winter severity as indicated by the freezing index will be noted. No consistent relationship between the amount of winter heave and freezing index, or between the amount of winter heave and frost depth, such as might be expected, is present in the data for the study period.

A further indication of the relationship between winter severity and the depth of frost penetration will be seen in Figure 26. In the upper part of this figure, daily mean temperatures are plotted for the coldest portions of the three winters of study. In the mid-portion of the figure, cumulative freezing degree-days are plotted. In the lower part of the figure, depths of frost penetration are plotted from data obtained at a typical test site. General relationships between the three displays, of the kind that would be anticipated, are readily observable.

DURABILITY OF SHOULDER BASE AND PAVEMENT SUBBASE MATERIALS

During the summer of 1969 a coring program was initiated to study principally the durability of the BAM, PAM and CAM materials as shoulder base and as pavement subbase. The program was completed in the summer of 1970. Close to 100 corings were made with a portable core drill in the CAM and PAM shoulder test sections and in the adjacent pavement subbase. A few additional corings were made in BAM and PCC shoulder sections. Concurrent corings in BAM and CAM pavement subbases of the Rehabilitated AASHO Road Test Project at Ottawa, Illinois, constructed in 1961, on a section of Interstate 80 east of the experimental shoulder test area where BAM pavement subbase was used, and at a location (the approaches to the Wolf Road overpass over I-80) where the pavement subbase was constructed of PAM material in 1965, provided additional durability information.

To classify the behavior of the materials, a rating was assigned to each coring site which described the condition of the core and of the wall of the core hole.

The rating included categories from 1 to 5. The core and the wall of the core hole were both rated and the average of the two ratings taken as the site rating. The following criteria were established for the ratings:

Rating	Criteria
1	Core surfaces and core hole wall smooth with no evidence of raveling.
2	Core surfaces and core hole wall in good condition except for slight raveling at junction of horizontal and vertical surfaces. Only slight durability deficiency indicated.
3	Core surfaces or core hole wall show appreciable evidence of raveling. Raveling may involve up to half the length of the core wall or core side surface. Core may have broken into a few large sections. Moderate durability deficiency indicated.
4	Core surfaces or core hole wall show general raveling. Core may have broken into a relatively large number of fragments which still show evidence of cementation individually. Severe durability deficiency indicated.
5	Total disintegration indicating complete absence of cementation.

Coring sites rated either 1 or 2 are considered to have demonstrated acceptable durability. Coring sites that received a rating of 3 or higher are considered to lack durability. Those rated 5 have undergone total breakdown of the cementing agent.

Corings were made through the shoulder at the pavement edge in both the sealed and unsealed pavement-shoulder joint sections, and through the pavement both at and between transverse cracks. Several corings also were made at mid-shoulder locations, and at the outer edge of the shoulders. Core site ratings were combined as average ratings for the materials. The results of the coring tests in the CAM and PAM materials as shoulder base and as pavement subbase on Interstate 80 at Joliet are shown in Table 12. It will be noted that, after two and three years of service, moderate to severe losses of durability as indicated by the core site ratings were limited to

TABLE 12

CORE SITE RATINGS

Material and Location	Shoulder Base	Pavement Subbase: Under Pavement	Pavement Subbase: Extension Under Shoulder
CAM - near sealed joint	4.2	1.2	2.9
CAM - near unsealed joint	4.2	1.3	3.6
PAM - near sealed joint	1.1	1.1	1.3
PAM - near unsealed joint	2.0	1.1	1.6

Note: Ratings shown are in each instance averages for either seven or eight cores.

the CAM shoulder base in the vicinity of the pavement edge. Slight losses of durability were recorded for this evidently critical location for the PAM materials. BAM and PCC coring sites, which are not recorded in the table, all showed ratings of 1.0.

The presence or absence of joint seal appeared to have little influence on core condition.

The PAM mixture appears to be more durable than the CAM as shoulder base, although a visual inspection of the cores taken from the PAM base at the pavement edge showed a slight but consistent tendency to ravel adjacent to the pavement-shoulder interface. Both the cores and core walls also showed a tendency to ravel at the top of the base adjacent to the bituminous surface, especially on the side next to the pavement.

Some additional information obtained from cores taken during the CAM and PAM coring operations is presented in Table 13. The compressive strengths of both the CAM and PAM cores included in the testing are indicative of reasonably strong materials. The amounts of material lost from recoverable cores in the standard freeze-thaw test appear to be acceptable. Little is known about the meaning of results from the freeze-thaw test that was conducted using brine, but it is obvious that the brine caused marked disintegration in materials which had shown good resistance in the standard test using distilled water.

The presence or absence of joint seal had little evident influence on the compressive strength and freeze-thaw test results.

Representative BAM, CAM, and PAM cores were soaked for a week in distilled water, weighed in the surface dry condition, and then oven dried to constant weight. It was found that the BAM cores had absorbed only half as much water as the PAM and CAM cores which had absorbed the equivalent of about 15% of their oven dry weight.

TABLE 13

STRENGTH AND DURABILITY OF CORES

Material and Location	Compressive Strength (psi)	Freeze-Thaw Weight Loss: Standard Test (percent)	Freeze-Thaw Weight Loss: 5 percent Brine (percent)
	Pavement Subbase		
CAM - near sealed joint	1321	5.6	57.5(6 cy.)
CAM - near unsealed joint	1858	6.4	54.6(5 cy.)
PAM - near sealed joint	992 2889 1037	8.6 11.8	37.7 30.3
PAM - near unsealed joint	1191 1374 2127 1196	9.5 10.9	42.7 25.1
	Shoulder Base		
CAM - near sealed joint	No testable core		
CAM - near unsealed joint	No testable core		
PAM - near sealed joint	1527 1050 1830 1139	9.5	26.5
PAM - near unsealed joint	619 1542 1637	11.3	58.1(10cy.)

Note: Tests were in accordance with ASTM standards, except that a five percent brine solution was used as the soaking agent on half of the freeze-thaw test cores. All freeze-thaw tests to 12 cycles except as noted.

The BAM was found to be more permeable than the PAM and CAM when tested by the constant-head method, but retained less water.

The durability studies are considered to indicate that CAM and PAM mixtures meeting the mixture-design criteria that governed those included in this investigation are not sufficiently durable to serve in shoulder bases under exposure conditions of the order experienced. These same mixtures, on the other hand, appear to have a reasonably good chance of affording satisfactory service in subbases under continuously reinforced pavement under the same conditions of exposure. The conclusion with respect to subbase usage was corroborated by additional core samplings during the same period in CAM subbase constructed in 1961 on I-80 at the AASHO Road Test site and in PAM subbase placed in 1965 on the approaches to the Wolf Road structure over I-80. It will be noted, however, that the conclusion is limited to usage under continuously reinforced pavement. At both the AASHO Road Test site and the Wolf Road site, some deterioration in cores taken from under transverse joints was evident.

Cores taken from both BAM shoulder bases and BAM pavement subbases on this project and others have shown no lack of resistance to freeze-thaw action. This is true also of the PCC shoulders.

SHOULDER CRACKING

Surveys of cracking in the outer shoulders were made during the spring and fall of each year following construction through 1969. The final survey, which was made in December 1969, covered the median shoulders as well as the outer shoulders.

For analytical purposes, the cracking that was observed in the shoulder surfaces during condition surveys was grouped into three categories:

(1) Transverse cracking

(2) Longitudinal cracking

  (a) near the pavement-shoulder joint

  (b) near the outer shoulder edge

  (c) in the mid-area of the shoulder

(3) Area cracking

  (a) alligator-type cracking

  (b) other complex cracking, or structural failure

Table 14 has been prepared to show the cracking observed in the final condition survey of December 1969 as related to the various shoulder paving materials. Transverse cracking is reported in number per 1000 lineal feet of shoulder; longitudinal cracking is reported in lineal feet per 1000 lineal feet of shoulder; and area cracking is reported in square feet per 1000 lineal feet of shoulder. The data are shown separately for the outer and median shoulders, and by shoulder subbase usage. It can be seen that: (1) the outer BAM shoulders have been subject to a considerable amount of area cracking; (2) the CAM and PAM base shoulders have been subject to a considerable amount of longitudinal and transverse cracking, and the PAM to some area cracking in addition; and (3) the PCC shoulders have been subject to a limited amount of transverse cracking even though contraction joints were installed at 20-foot intervals. Longitudinal and area cracking typical of that occurring in the bituminous concrete surfacing over the CAM base are shown in Figure 27; a typical BAM failure is shown in Figure 28.

Except in one test section of BAM on earth subgrade where an obvious lift separation and attendant structural failure were widespread, cracking in the outer BAM shoulders has been confined mainly to the outer one half of the shoulder area where the tapering section thins.

TABLE 14

RELATION OF SUBBASE USE TO SHOULDER CRACKING BY TYPE OF SHOULDER BASE MATERIAL
December 1969

Base Material	Subbase Type 1/	Transverse Shoulder Cracks 2/		Longitudinal Shoulder Cracks 3/						Shoulder Area-Cracking 4/	
				Inner Edge		Mid Shoulder		Outer Edge			
		Outer	Median	Outer	Median	Outer	Median	Outer	Median	Outer	Median
BAM	A	0	0	0	0	15	0	0	0	245	0
	B	<1	0	0	0	0	2	10	6	496	0
	C	0	0	0	0	10	6	1	0	207	0
	E	0	2	0	0	2	0	0	0	1617	0
CAM	A	13	9	228	496	0	0	19	0	0	0
	B	17	22	231	475	0	0	26	54	0	0
	C	15	33	527	377	10	0	116	0	0	0
	E	11	10	284	604	7	0	116	0	0	0
PAM	A	27	23	429	27	207	0	46	0	105	0
	B	29	18	511	479	51	0	222	152	33	7
	C	27	14	323	518	242	0	294	342	12	0
	E	27	19	442	200	154	0	335	0	237	16
PCC	A	9	10	0	0	5	0	0	0	0	0
	B	4	4	0	0	0	0	0	0	0	0
	C	6	3	0	0	1	0	0	0	0	0
	E	9	3	0	0	0	0	0	0	0	0

1/ Type A Subbase is similar to concrete sand; Type B is Illinois Class X aggregate for concrete; Type C is Illinois Size B aggregate for concrete; and Type E is no subbase.

2/ Number of transverse cracks per 1000 lineal feet of shoulder

3/ Lineal feet of longitudinal cracking in 1000 lineal feet of shoulder.

4/ Square feet of area cracking in 1000 lineal feet of shoulder.

Figure 27. Longitudinal and associated area cracking in bituminous concrete surfacing on CAM base.

Figure 28. Area or alligator cracking in outer wheelpath portion of BAM shoulder.

The longitudinal cracking that has occurred near the pavement edge over major portions of the CAM and PAM shoulders lies within a few inches to a foot of the pavement edge. The area cracking in the PAM shoulders has taken place in the space between the pavement edge and the longitudinal crack that has formed close to the edge.

This longitudinal cracking in the CAM and PAM shoulders appeared to be associated consistently with losses in durability in the cemented base materials. While a causal relationship was not proven, it can be strongly suspected that the longitudinal cracking and attendant area cracking have been induced by traffic loadings where base support has been lost. Support to this belief was obtained from an observation of a CAM shoulder on another project that had undergone one winter of exposure but had not been opened to traffic. The pavement-shoulder joint had opened and the CAM shoulder base had started deteriorating along its exposed vertical face and at the interface with the shoulder surfacing, leaving the shoulder surfacing unsupported at the pavement edge. While the shoulder surfacing was almost entirely free of longitudinal cracking , one passage of a wheel would have caused longitudinal cracking to develop near the pavement edge for the entire length of the shoulder. Since opening of the shoulder-pavement joint invariably occurs, except for concrete shoulders tied to the pavement slab, this observation strongly suggests that a paved shoulder base to be adequate must be one that is sufficiently strong and durable that it can support traffic loadings while standing unsupported on a vertical face.

The cracking behavior of the various shoulder types does not appear to be closely related to the shoulder subbase material type, or to the presence or absence of shoulder subbase, except that the alligator cracking experienced by the BAM shoulders is significantly greater in amount where no subbase was present.

It is obvious from Table 14 that the PCC shoulders on the experimental project are in a distinctly better condition than any of the other types under observation. In addition, tying the concrete shoulders to the 8-inch CRC pavement had no effect on the pattern of transverse cracking in the pavement slab. Crack counts were made in randomly selected 500-foot-long sections of pavement at four locations in the PCC and CAM shoulder test sections and at two locations in the BAM and PAM shoulder test sections. The average intervals between transverse cracks in the CRC pavement for the various shoulder test sections were 3.0 feet for the PAM, 3.2 feet for the CAM, and 3.3 feet for the BAM and PCC. A typical view of the PCC shoulders is shown in Figure 29.

The results of an investigation of possible relationships between pavement-shoulder joint treatment and shoulder cracking are presented in Table 15. In all instances but one, it would appear that sawing and sealing the pavement-shoulder joint, as compared with no special joint treatment, had little influence on shoulder cracking. The single exception occurred in the CAM and PAM shoulder base sections where consistently less longitudinal cracking occurred near the pavement-shoulder joint where the sawing and sealing treatment was used.

The widespread occurrence of cracking in the shoulders with CAM and PAM bases suggests that CAM and PAM mixtures, at least in the formulations studied, are not appropriate in this usage under the environmental conditions experienced in the experimental area. The BAM shoulders also did not perform well in the experimental section, which is at variance with rather broad experience with this type of shoulder in regular construction elsewhere in Illinois. The predominant type of failure in the BAM shoulders (alligator cracking, accompanied by depression) suggests an inadequacy of the structure in supporting the applied loads. The PCC shoulders, although showing a small amount of intermediate cracking in the constructed 20-foot

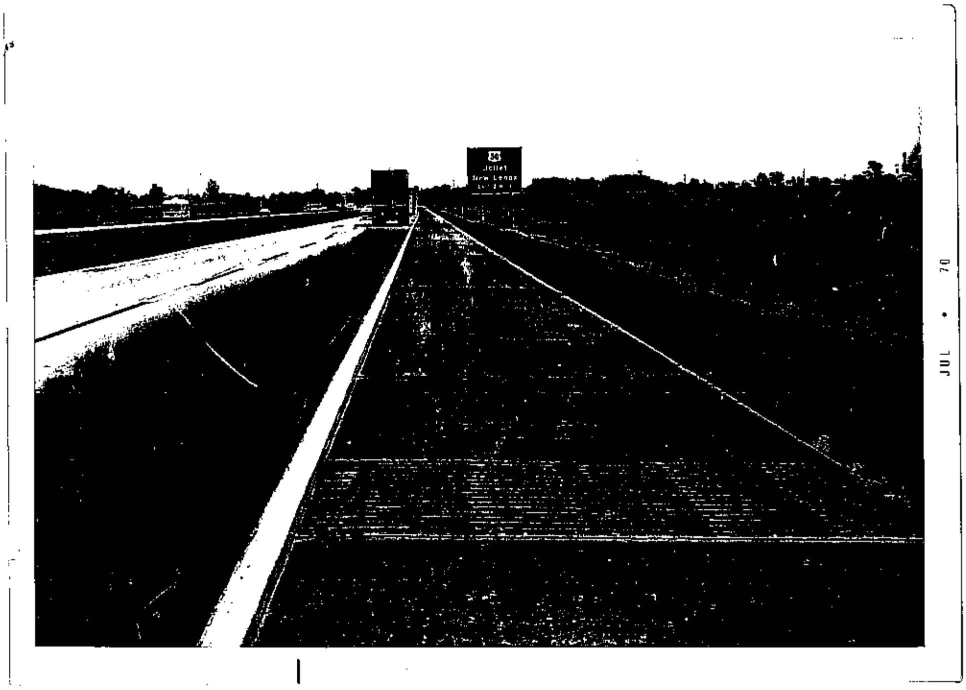


Figure 29. Outer portland cement concrete shoulder adjacent to eastbound pavement.

TABLE 15

RELATION OF PAVEMENT-SHOULDER JOINT TREATMENT TO SHOULDER CRACKING
December 1969

Base Material	Subbase Type 1/	Transverse 2/ Shoulder Cracks		Longitudinal Shoulder Cracks 3/ Inner Edge		Mid Shoulder		Outer Edge		Shoulder 4/ Area-Cracking	
		Sealed	Unsealed	Sealed	Unsealed	Sealed	Unsealed	Sealed	Unsealed	Sealed	Unsealed
BAM	A	0	0	0	0	0	15	0	0	13	282
	B	<1	<1	0	0	0	2	6	10	48	448
	C	0	0	0	0	16	0	1	0	207	0
	E	0	0	0	0	0	2	0	0	645	972
CAM	A	12	10	68	656	0	0	0	19	0	0
	B	17	21	201	505	0	0	39	41	0	0
	C	24	24	200	704	10	0	79	37	0	0
	E	8	13	267	621	0	7	58	31	0	0
PAM	A	24	25	47	410	20	187	46	0	101	4
	B	26	21	312	689	3	49	138	236	21	19
	C	19	22	232	609	39	203	390	247	0	12
	E	22	24	191	452	86	68	150	185	15	138
PCC	A	0	0	0	0	6	0	0	0	0	0
	B	<1	<1	0	0	0	0	0	0	0	0
	C	0	0	0	0	0	0	0	0	0	0
	E	2	0	0	0	0	0	0	0	0	0

1/ Type A Subbase is similar to concrete sand; Type B is Illinois Class X aggregate for concrete; Type C is Illinois Size B aggregate for concrete; and Type E is no subbase.

2/ Number of transverse cracks per 1000 lineal feet of shoulder.

3/ Lineal feet of longitudinal cracking in 1000 lineal feet of shoulder.

4/ Square feet of area cracking per 1000 lineal feet of shoulder.

panels, can be considered at this time to be showing acceptable performance, and to be serving significantly better than any of the other types included in the experiment.

RESULTS AND RECOMMENDATIONS

During the three-year period of study, the paved shoulders of the experimental section of I-80 east of Joliet did not show strong upward differential movement relative to the adjoining pavement. This is at variance with the experiences elsewhere in northern Illinois that were major factors in causing the I-80 experimentation to be undertaken. Shoulder-pavement designs that had shown major differential movements are well represented in the I-80 experimentation. The subgrade soil is very similar to that at the other locations with respect to frost susceptibility. The major difference in the total roadway design between the experimental section and the sections that were constructed earlier was the change from the use of an unstabilized granular subbase material under the mainline pavement to the use of granular mixtures stabilized with asphalt, cement, and lime-flyash. It is believed that this has removed an important source of water that contributed to the earlier problems.

The structural performance of the bituminous-aggregate mixture (BAM) shoulders on the I-80 experimental project has been inferior to that which has been experienced elsewhere in Illinois. It is strongly suspected that construction aberrations unique to the experimental project were responsible mostly for the poor showing of the BAM as a structural material. Over a considerable portion of the areas where outright failure took place, distinct planes of separation were found between construction lifts. The presence of a soil film on the layer interfaces, which construction records reveal probably formed because of a considerable time-lapse between the placing of layers, is believed to have been a major contributor to the failures.

It was further observed that much of the structural failure in the BAM sections took place near the outer edges where the structural design called for thicknesses of 6-7 inches, and where constructed thicknesses have been noted to be below plan thicknesses. Whether or not construction to full-plan thickness would have prevented the structural failures can only be conjectured upon, but it appears certain that little reserve strength would exist even at full thickness. An increase in thickness under similar conditions of support in future construction is indicated.

The shoulder pavements that include cement-aggregate mixtures (CAM) and pozzolan-aggregate mixtures (PAM) are showing an extensive amount of the longitudinal cracking in the bituminous concrete surfacing near the pavement-shoulder joint that was found prevalent previously in shoulders where these mixtures were used. This cracking is suspected of being traffic-induced where base support has been lost through durability failure. The experimentation thus far has not suggested ways of avoiding this longitudinal cracking.

The performance of the portland cement concrete (PAC) shoulders is significantly better than that of any of the other types. Tiebars appear to be a desirable feature that can be used to keep the shoulder-pavement joint closed only in connection with PCC shoulders. While service-life projections of perhaps 20 years based on three years of service cannot be made with a truly high degree of confidence, it would seem that of the various types of paved shoulder included in the experiment, the PCC shoulders may have the best chance of serving the longest time without need for special maintenance and can be considered as a satisfactory alternative paved shoulder type.

The presence of the open-graded shoulder subbase materials, as compared with the absence of subbase, contributed to significantly better performance of the BAM shoulders. Fewer transverse cracks also were observed where the subbase was used under the PCC shoulders. No consistent improvement that might be attributable to the use of subbase was noted in connection with the CAM and PAM base shoulders. All shoulder subbase materials were extended to the side slopes for drainage. None of the three subbase materials appeared to serve distinctly better than the others. Problems encountered in keeping the sand subbase in place during succeeding construction operations discourage its further use as normal practice.

The hot-poured rubber-asphalt joint sealant is retarding the development of longitudinal cracking at the pavement-shoulder joint of the CAM and PAM sections. It has had no measurable effect on the behavior of the BAM and PCC shoulders up to the present time. More experience is needed before any possible beneficial effect when used with BAM and PCC shoulders can be determined.

No evidence was observed to indicate that any of the three mixtures, BAM, CAM, or PAM, was not serving adequately as a pavement subbase material. Cores taken from the subbase at under-pavement locations showed adequate durability in all three mixtures.

The following additional observations were made:

Very dry and very dense conditions can exist in the soil immediately below subgrade level at the time of placement of shoulder structures. This suggests a potential for differential heave of the shoulder with respect to the pavement through moisture gain if the subgrade soil under the pavement is of appreciably higher moisture content.

The pavement and shoulders together move upward when frost penetrates the earth subgrade. Observed movements were mostly of the order of 0.02 to 0.04 ft. although greater movements were sometimes recorded. The shoulders exhibited a tendency to show greater movement than the pavement.

IMPLEMENTATION

Based on the results of this research, corroborated in some instances by additional observations made elsewhere in the State, the Illinois Department of Transportation has:

(1) Continued to use the Bituminous-Aggregate Mixture (BAM) as a shoulder material.

(2) Increased the specified thickness of BAM shoulders at the outer edge by one inch, and revised specifications for thickness tolerance to assure less deviation from the intended thickness.

(3) Continued to use an open-graded drainage course under its shoulder structures.

(4) Rejected Cement-Aggregate Mixture (CAM) and Pozzolan-Aggregate Mixture (PAM) as shoulder-base materials.

(5) Accepted CAM and PAM as pavement subbase material alternatives along with the previously used BAM.

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